**US Army Corps
of Engineers**
Waterways Experiment
Station

HYDRAULICS LAB COPY

Yellow Creek Sedimentation Study

Numerical Model Investigation

by   Ronald R. Copeland
     Hydraulics Laboratory

The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.

Yellow Creek Sedimentation Study

Numerical Model Investigation

by   Ronald R. Copeland

   Hydraulics Laboratory

   U.S. Army Corps of Engineers
   Waterways Experiment Station
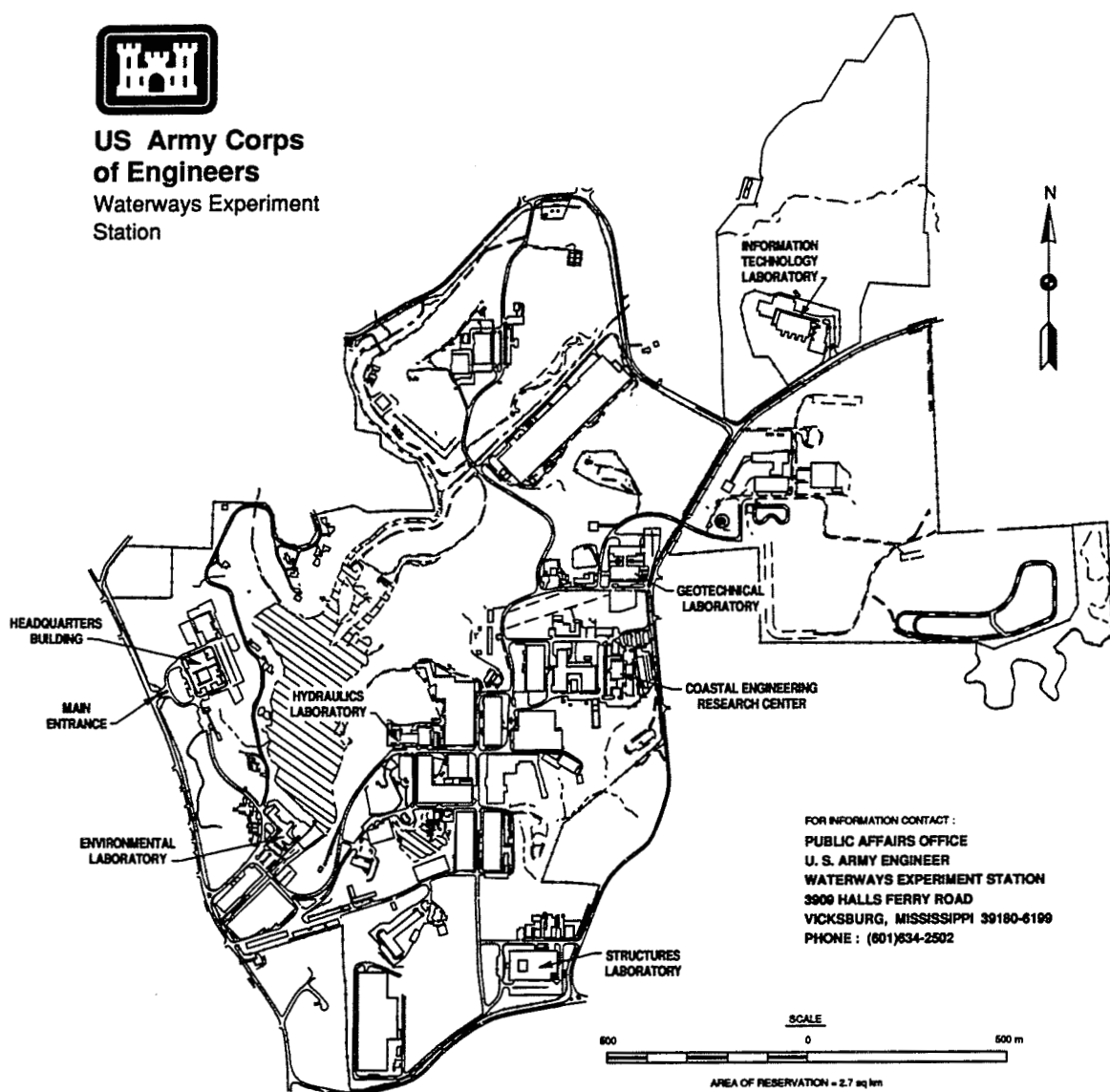   3909 Halls Ferry Road
   Vicksburg, MS   39180-6199

Final report

**US Army Corps
of Engineers**
Waterways Experiment
Station

N

INFORMATION
TECHNOLOGY
LABORATORY

GEOTECHNICAL
LABORATORY

HEADQUARTERS
BUILDING

HYDRAULICS
LABORATORY

MAIN
ENTRANCE

COASTAL ENGINEERING
RESEARCH CENTER

ENVIRONMENTAL
LABORATORY

FOR INFORMATION CONTACT :
PUBLIC AFFAIRS OFFICE
U. S. ARMY ENGINEER
WATERWAYS EXPERIMENT STATION
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199
PHONE : (601)634-2502

STRUCTURES
LABORATORY

SCALE

500   0   500 m

AREA OF RESERVATION = 2.7 sq km

Contents

List of Figures

Preface

The numerical model investigation of potential aggradation and degradation in the proposed Yellow Creek channel improvement project, located near Middlesboro, KY, was conducted at the U.S. Army Engineer Waterways Experiment Station (WES) at the request of the U.S. Army Engineer District, Nashville (ORN).

This investigation was conducted during the period January 1989 to August 1992 in the Hydraulics Laboratory of WES under the direction of Messrs. Frank A. Herrmann, Jr., Director of the Hydraulics Laboratory, WES; R. A. Sager, Assistant Director of the Hydraulics Laboratory; Marden B. Boyd, Chief of the Waterways Division (WD), Hydraulics Laboratory; and Michael J. Trawle, Chief of the Math Modeling Branch (MMB), WD. Mr. William A. Thomas, WD, provided general guidance and review. The project engineer and author of this report was Dr. Ronald R. Copeland, MMB. Technical assistance was provided by Ms. Brenda L. Martin and Mrs. Peggy H. Hoffman, MMB.

During the course of this study, close working contact was maintained with Mr. J. David Hendrix, ORN, who served as coordinating engineer, providing required data, technical assistance, and review.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
square miles	2.589998	square kilometers
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

The Prototype

Yellow Creek drains approximately 103 square miles[1] of the Cumberland Mountains in southeastern Kentucky (Figure 1). Most of the drainage basin is covered by forests. Strip and pit coal mining is prevalent in the basin. The major tributaries to Yellow Creek are Little Yellow Creek, with a drainage area of 11.8 square miles; Bennetts Fork, with a drainage area of 13.5 square miles; and Stony Fork, with a drainage area of 16.4 square miles. These tributaries are characterized by steep slopes and boulder beds and converge in a cup-shaped valley in which the city of Middlesboro rests. Middlesboro's water supply reservoir is located in the Little Yellow Creek drainage basin, affecting runoff from 7.1 square miles of the basin. Originally, Bennetts Fork and Stony Fork joined to form Yellow Creek southwest of Middlesboro. A bypass channel constructed by the U.S. Army Corps of Engineers in 1939 collects the flow from these tributaries and diverts it northwesterly around Middlesboro and back into Yellow Creek, which then flows north into the Cumberland River.

The city of Middlesboro was founded in 1889 as part of a development plan to establish a major iron and coal industrial center. Flooding has been a problem in the city since its founding. The first flood protection measures on Yellow Creek were constructed in 1890, when the channel was straightened and enlarged through the center of town. However, this project did little to alleviate the flood problems.

In 1939, the Corps of Engineers constructed the bypass channel and levee system that diverts the headwaters of Yellow Creek around Middlesboro. This project did not completely eliminate flooding problems due to insufficient channel capacity in Yellow Creek downstream from the bypass channel confluence. Backwater from this reach retards the flow and causes flooding in the lower part of the city.

---

[1] A table of factors for converting non-SI units of measurement to SI units is found on page vi.

VICINITY MAP

PINEVILLE
CUMBERLAND RIVER

SCALE
0   10,000 FT

KENTUCKY

YELLOW CREEK

DIVERSION
CANAL

MIDDLESBORO

CUMBERLAND MOUNTAINS

VIRGINIA

CUMBERLAND
GAP

TENNESSEE

N

LEGEND

●	GAGE
∿	RIVER
—	STREETS
⋯	LEVEE
⌇	RAILROAD
× 14	RIVER MILE

N

YELLOW CREEK

BYPASS CHANNEL

CONCRETE CHUTE

STONY FORK

LITTLE YELLOW
CREEK

BENNETTS FORK

MIDDLESBORO

FERN LAKE

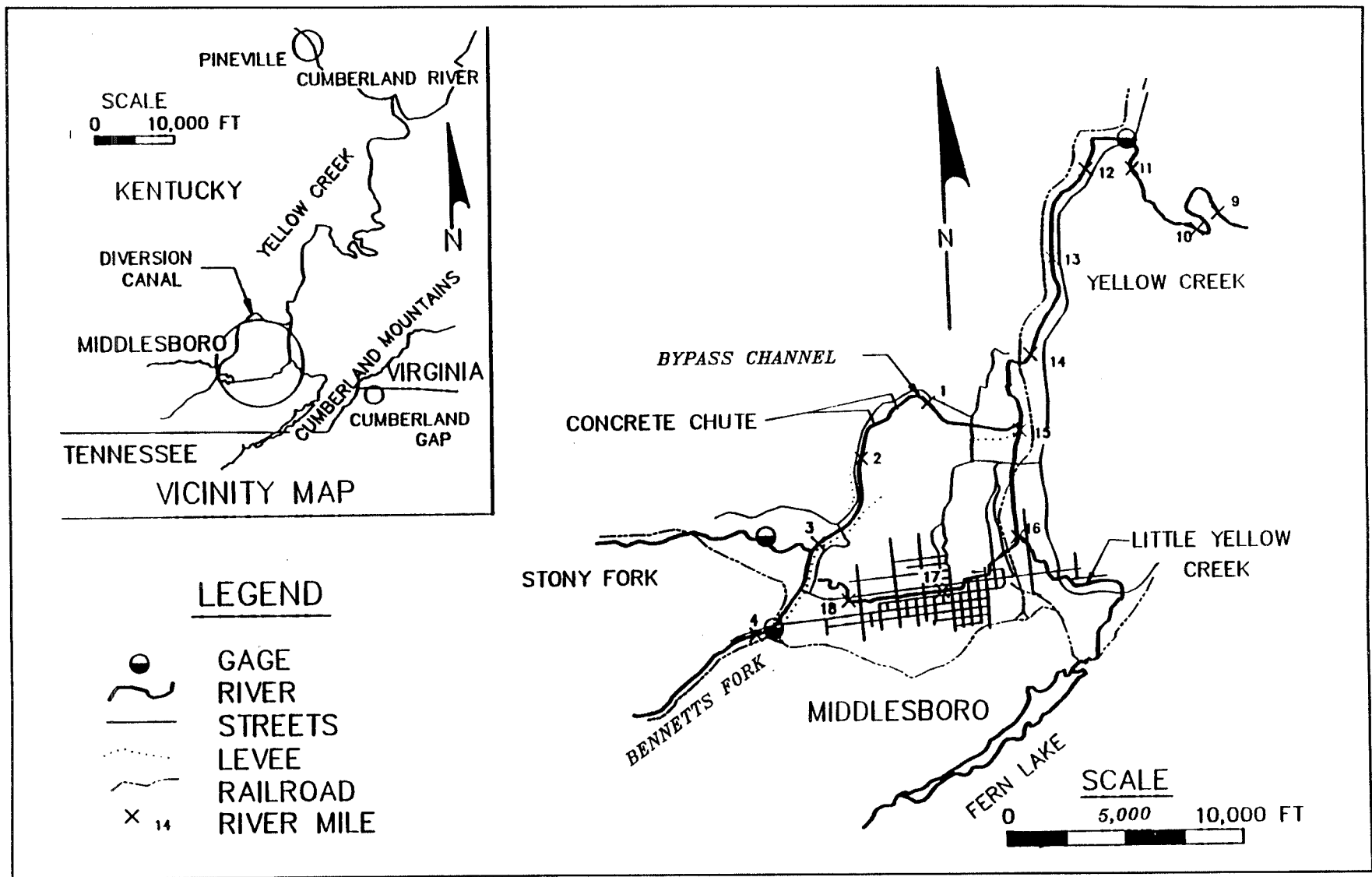SCALE
0   5,000   10,000 FT

Figure 1. Location and vicinity maps

In 1952 the Corps of Engineers performed clearing and snagging maintenance on about 3.8 miles of the natural section of Yellow Creek below the bypass channel. The clearing of heavy timber growth on the banks and removal of boulders, gravel deposits, and debris from the channel significantly improved conditions initially. However, due to regrowth of brush and trees, the benefit from this project has been reduced and backwater flooding still occurs in the lower part of the city. A 1971 Flood Plain Information Report[1] showed substantial portions of Middlesboro subject to flooding from the 100-year and Standard Project floods.

The numerical model study reach extends from mile 9.43 on Yellow Creek, up Yellow Creek to mile 14.96, which is the bypass channel confluence, and then up the bypass channel 3.9 miles to the mouth of Bennetts Fork. Downstream from mile 13.1, Yellow Creek flows through a very narrow valley bordered on both sides by rugged hills that rise more than 1,000 ft above the valley floor. In this reach, the slope is about 0.0011, and the streambed alternates between bedrock, gravel riffles, and coarse sand. Upstream from mile 13.1, Yellow Creek has a narrow but distinct floodplain. In this reach, channel bottom width ranges between 30 and 60 ft, banks are about 10-15 ft high, and the slope is about 0.0005. The streambed alternates in a riffle and pool sequence, with gravel on the riffles and coarse sand in the pools. The bypass channel enters Yellow Creek at mile 14.96 and has a bottom width of about 30 ft at the confluence, increasing to about 100 ft at mile 1.37. This reach of the bypass channel has experienced considerable degradation since its construction in 1939. At the upstream end of this reach, downstream from a supercritical flow concrete chute, the streambed has scoured to shale bedrock. Downstream from the exposed bedrock, the bypass channel has a sand and gravel bed. A comparison of surveys taken in 1971 and 1980 indicate that the reach may have stabilized at a slope of about 0.005 (Figure 2). A concrete chute, constructed in a cut section between bypass channel miles 1.68 and 1.38, has a variable bottom width between 132 and 105 ft. It controls upstream water-surface and bed elevations with a critical depth control at its inlet. Upstream of the concrete chute, between miles 1.68 and 3.9, the bypass channel has an average bottom width of 100 ft and a slope of about 0.0028. A low-flow channel has a sand and gravel bed, which becomes finer in a downstream direction. Channel bars and benches, formed from stream deposits, are composed primarily of fine sand and silt.

Purpose of the Numerical Model Study

Siltation occurs in the upstream portion of the bypass channel and has been

---

[1] U.S. Army Engineer District, Nashville. (1971). "Flood plain information; Yellow Creek and Little Yellow Creek, Bennetts and Stony Forks, and Diversion Canal; Middlesboro, Kentucky," Prepared for the city of Middlesboro by the U.S. Army Engineer District, Nashville, Nashville, TN.
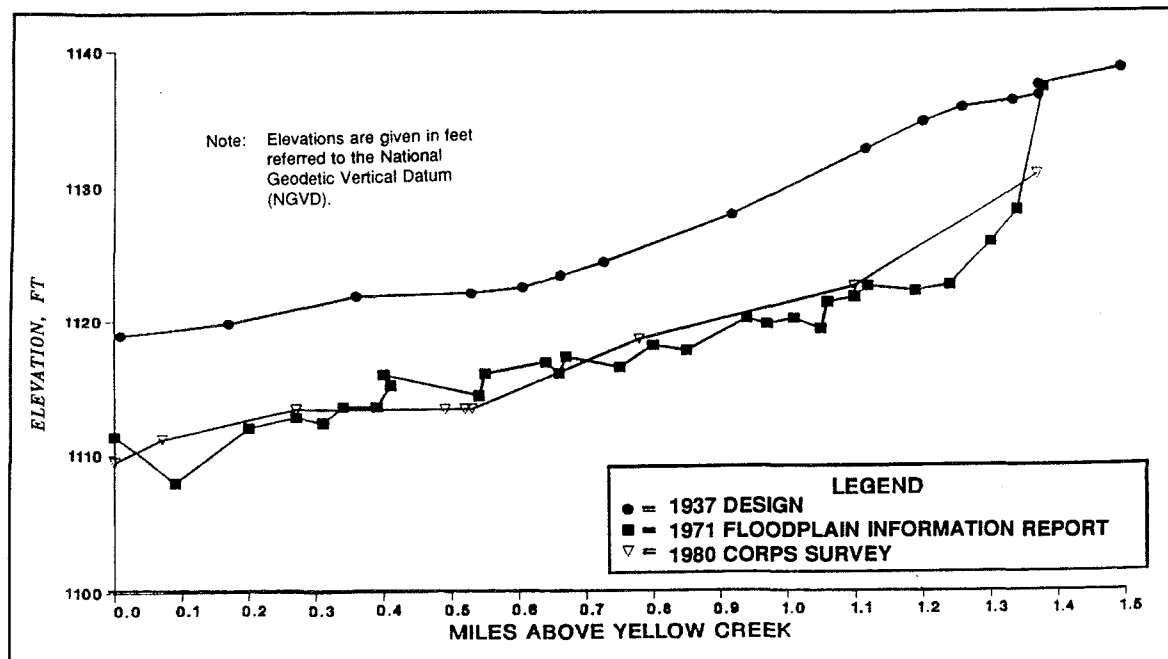
Figure 2. Historical thalweg profiles in bypass channel, downstream from concrete chute

attributed to strip mining and construction of haul roads in the basin.[1] Following floods in 1963 and 1965, sediment deposits were removed from the bypass channel to maintain the design capacity. After a flood in 1977, about 50,000 cu yd of channel deposits were removed in a second dredging operation. Between 1978 and 1987, approximately 36,000 cu yd of sediment were deposited. A resurvey in January 1992 indicated that between 1987 and 1992, 33,800 cu yd were deposited upstream from mile 2.1 and 11,400 cu yd were eroded between the concrete chute and mile 2.1. One of the purposes of this study was to develop a model that could simulate historical sediment deposition in the bypass channel. This model would then be used to evaluate various dredging strategies.

The Corps of Engineers is seeking to reduce backwater flooding problems in the city of Middlesboro by increasing channel capacity in the reach of Yellow Creek below the bypass channel. A second purpose of this investigation is to evaluate potential for sediment deposition or scour with the proposed channel enlargement design and to investigate alternate enlargement designs, if appropriate.

---

[1] U.S. Army Engineer District, Nashville. (1966). "Cumberland River Basin, Middlesboro, Kentucky, local protection project - Design Memorandum No. 1," Nashville, TN.

2   The Model

Description

The TABS-1 one-dimensional sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William A. Thomas at the U.S. Army Engineer District, Little Rock, in 1967. Further development at the U.S. Army Engineer Hydrologic Engineering Center (USAEHEC) by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1991). Additional modification and enhancement to the basic program by Mr. Thomas at the U.S. Army Engineer Waterways Experiment Station (WES) led to the TABS-1 program currently in use. The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effects on the sediment transport capacity at cross sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow. Critical depth is assigned for water-surface elevation if the backwater calculations indicate transitions to supercritical flow. However, for supercritical flow, hydraulic parameters for sediment transport are calculated assuming normal depth in the channel. A more detailed description of the program capabilities is found in Appendix A.

The Hydraulic Design Package for Flood Control Channels, SAM (Thomas et al., in preparation), is a set of hydraulic computer programs for preliminary design of channels. The purpose is to calculate the width, depth, slope, and roughness for stable channels in alluvial material. SAM is being developed at WES as part of the Flood Control Channels Research Program. Beta test versions have been released to several Corps of Engineer Districts for testing. SAM is composed of three modules. The first module can be used for hydraulic calculations and for determining stable channel dimensions. This includes calculation of hydraulic parameters for uniform flow in both simple and complex channels and determination of width, depth, and slope using analytical equations for sediment transport and roughness. The second module can be used to select an appropriate sediment transport equation based on existing channel conditions, and to calculate a sediment transport rating curve. The

third module calculates sediment yield by integrating the sediment discharge and flow duration curves. Used in sequence, these modules can be used to quickly evaluate stability of proposed channels by determining aggradation and degradation potential. In this fashion various design alternatives may be compared, and maintenance requirements identified.

Channel Geometry

TABS-1 numerical models of two different reaches were developed during the investigation. The downstream model included 5.53 miles of Yellow Creek between miles 9.43 and 14.96 and then 1.37 miles of the bypass channel from its confluence with Yellow Creek to the downstream end of the concrete chute. Initial geometry for this model was taken from 1980 Corps of Engineer surveys. The upstream model included the bypass channel upstream from the concrete chute (mile 1.7) to the mouth of Bennetts Fork (mile 3.9). Initial channel geometry for this model was taken from Corps of Engineer surveys taken after removal of sediment deposits in 1978. During the course of the study, the 1978 geometry was modified to include a low-flow channel. The average bed elevations in the modified cross sections were the same as for the 1978 cross sections. The base width and depth of the low-flow channel were based on the actual low-flow channel that was identified from 1987 surveys. These 1987 surveys were used to determine sediment accumulation between 1978 and 1987 in the prototype, and as initial geometry to calculate sediment deposition between 1987 and 1992. The channel was resurveyed in January 1992, and these data were used to determine prototype deposition. Numerical model cross-section locations and model boundaries are shown in Figure 3.

Hydrographs

Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that changes in bed elevation due to deposition or scour do not significantly change the hydraulic parameters during that event. At relatively high discharges, durations need to be short; time intervals as low as 8 hours were used for flood peaks in the historical histograph. Time-steps of 15 minutes were used for the Standard Project Flood. At low discharges, the time interval may be extended. Time intervals up to 6 days were used during the historical simulations.

A hydrograph simulated by a series of steady-state events of varying durations is called a histograph. The historical histograph used in the numerical model was based on data from the U.S. Geological Survey's (USGS) gage on Yellow Creek. The gage is located at mile 11.4; prior to 1970 it was located at mile 13.1. Mean daily discharges greater than 200 cfs were used to develop a historical histograph between January 1978 and September 1991. Sediment transport was found to be negligible when the discharge on Yellow Creek was less than 200 cfs. In addition to mean daily flows, 20 peak discharges greater

Figure 3. Cross-section locations
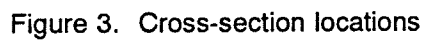
than 3,000 cfs were reported between 1978 and 1991. Mean daily flows were adjusted to account for the increased sediment transport potential at high-flow events. Reported peaks were assigned a duration of 8 hours and the corresponding mean daily flow was reduced to maintain the same runoff volume. The 8-hour duration was chosen based on durations of actual flood hydrographs measured in July 1965 and July 1967.[1] The 14-year historical histograph is shown in Plate 1. The abscissa on this plate is discontinuous because discharges less than 200 cfs were excluded.

Tributary inflow distributions for the historical and the Standard Project Flood histographs were determined from the Standard Project Flood peak flow percentages.[1] The discharge in the diversion canal upstream from the Yellow Creek confluence was 89 percent of the flow in Yellow Creek at the gage. In the diversion canal upstream from Stony Fork, the discharge was 51 percent of the Yellow Creek flow.

An average annual flow histograph was used to determine average annual deposition. USGS discharge duration data were used to develop this histograph. The period of record was 46 years (1941-1987). The data included discharges as high as 6,300 cfs. Discharge durations for larger events were estimated using seven peak discharges, greater than 7,000 cfs, that occurred between 1941 and 1987, assuming that their hydrograph shape was similar to the July 1965 flood hydrograph. This allowed for inclusion of discharges up to 11,000 cfs. An annual flow duration histograph was developed by assuming a symmetrical shape, and is compared to the 1978 annual hydrograph in Plate 2. This plate demonstrates that approximately equal volumes of runoff occur for both hydrographs, but distributions are significantly different.

Downstream Water-Surface Elevation

Water-surface elevation rating curves at the downstream boundary of the numerical model were taken from HEC-2 backwater calculations provided by the U.S. Army Engineer District, Nashville. At mile 9.43 on Yellow Creek, normal depth was assumed. On the bypass channel, backwater calculations upstream from the concrete chute defined the water-surface elevation at mile 1.70.

Bed Material

Bed material samples were collected from Yellow Creek and the bypass channel in March 1989 by engineers from WES and Nashville District. These samples indicated a wide variation in sizes (Plates 3-5). The stream channel is composed of two distinct classes of material. The low-flow channel in the

---

[1]   U.S. Army Engineer District, Nashville, 1971.

bypass channel is composed primarily of sand and gravel, while the bars, banks, and benches are composed primarily of fine sand and silt. Channel surveys of the bypass channel indicated that deposition occurs primarily on the bars and benches. Field observations in Yellow Creek downstream of the bypass channel indicated that fine sediments were depositing in slack-water areas, along the banks and behind vegetation. It was also observed during the field investigation that several portions of Yellow Creek are armored with large flat cobbles. These alternate with sections of coarse sand and gravel beds in typical riffle-pool sequences. Surveys were not extensive enough to identify all of the riffles and pools. Therefore, average bed gradations were used in the numerical model. Some sections, identified as riffles during model adjustment, were assigned immobile beds to prevent excessive scour in the model.

Channel Roughness

Hydraulic roughness is influenced by grain size, bed form, water depth, bank roughness, changes in channel shape, and changes in flow direction or concentration of flow due to bends and confluences. In the one-dimensional numerical model these effects are accounted for by the Manning's roughness coefficient. The roughness coefficient may vary significantly with discharge and time. The influence of grain roughness is known to decrease with increases in depth. Resistance due to bed forms can decline dramatically when dunes are washed out and replaced by a plane bed or antidunes. Greater momentum at high flows increases resistance due to channel bends and confluences. Local scour at high flow also tends to make channel cross sections more irregular, increasing roughness. In Yellow Creek and the bypass channel, the effects of vegetation on the banks are more significant than other factors in determining total channel roughness. High-water marks from events of known magnitude and hydraulic geometry are frequently used to estimate roughness coefficients in cases where vegetation plays such a significant role. High-water marks from the 11,300-cfs peak flood of April 1977 were used by the Nashville District to estimate Manning's roughness coefficients for existing channel conditions in their HEC-2 backwater model. Channel roughness coefficients ranged between 0.030 and 0.043.

Adjustment of the HEC-2 roughness coefficients was required in the TABS-1 numerical model. Overbank roughness coefficients in the HEC-2 model varied laterally; and since this option is not available in the TABS-1 model, average overbank coefficients were determined. Also, bridge losses are not calculated in the TABS-1 model and must be approximated by increases in roughness coefficients. This was done at the railroad bridge at mile 14.15 on Yellow Creek. Losses at other bridges were found to have an insignificant effect on sediment transport. Further adjustments to channel roughness coefficients were required in the TABS-1 model to account for loss of channel conveyance area due to deposition. This was accomplished by running a series of steady-state discharges for short durations in the TABS-1 model and comparing calculated water-surface elevations to those from the HEC-2 model. The

average channel roughness coefficient was 0.036 in the TABS-1 model compared to 0.038 in the HEC-2 model. With the roughness coefficient adjustment, calculated water-surface elevations in the TABS-1 and HEC-2 models were within 0.1 ft.

Sediment Inflow

Available sediment inflow measurements were inadequate to define sediment inflow for the entire range of historical and design discharges. Therefore sediment inflow was used as an adjustment parameter in the numerical models. An initial estimate of sediment inflow to the upstream model was determined by calculating average sediment transport capacity at the five cross sections farthest upstream. Sediment inflow was then reduced or increased by equal percentages in the adjustment phase of the study. Calculated outflow from the upstream model was used as inflow to the downstream model.

Sediment inflow concentrations from Stony and Bennetts Forks were taken to be the same because of the drainage basins' similarities. Sediment inflow from Little Yellow Creek is unknown, but is assumed to be considerably less than the bypass channel tributaries' drainage because of the effects of Fern Lake dam. Sediment inflow concentrations were arbitrarily taken to be 50 percent of those at Stony and Bennetts Forks. The sensitivity of this assumption was tested during the adjustment phase of the study.

Transport Function

Five transport functions were considered for use in this study: Ackers-White (1973); Laursen-Madden (USAEHEC 1991); Meyer-Peter and Müller (M-PM) (1949); Toffaleti (1968); Yang (1973, 1984); a combination of the Toffaleti and Meyer-Peter and Müller equations; and the Laursen-Copeland function, which is a modification of the Laursen function (Laursen 1958) developed for sand and gravel streams (Copeland and Thomas 1989). Without adequate measured data, it is not possible to effectively evaluate the applicability of the various functions. Average calculated sediment transport potential of very fine sand (VFS) from the five upstream cross-sections in the diversion canal are shown in Plate 6. The Ackers-White and Meyer-Peter and Müller functions produced results considerably different from the other functions and were not given further consideration. Although the Toffaleti and Laursen-Madden functions are known to be appropriate for sand-bed streams, their application to coarse sand and gravel streams is unsatisfactory. The Yang functions were developed for single grain size analysis, and may produce discontinuous results when applied in a multiple grain size analysis. The combined Toffaleti and Meyer-Peter and Müller and the Laursen-Copeland functions have been applied successfully to sand and gravel streams similar to Yellow Creek.

The Laursen-Copeland sediment transport function was used in this study. The new transport function was developed for streams with both fine and coarse sediment transport. It has been applied to numerical model studies on Corte Madera Creek in California (Copeland and Thomas 1989) and the Waimea River in Hawaii (Copeland 1990). The Laursen-Copeland function incorporates data for transport of gravels in addition to the sand data used to develop the original Laursen function. There are also differences in the way hydraulic parameters are calculated.

Laursen (1958) calculated grain shear stress using the mean depth of flow:

$$\tau' = \frac{\rho V^2}{58} \left(\frac{d_{50}}{D} \right)^{\frac{1}{3}} \tag{1}$$

where

τ' = grain shear stress
ρ = water density
V = average velocity
d_{50} = particle size of which 50 percent of the bed is finer
D = mean flow depth

Mean flow depth is replaced by the hydraulic radius due to grain roughness, R_b' in the Laursen-Copeland function:

$$\tau' = \frac{\rho V^2}{58} \left(\frac{d_{50}}{R_b'} \right)^{\frac{1}{3}} \tag{2}$$

R_b' is calculated using the Limerinos (1970) equation, as restructured by Burkham and Dawdy (1976):

$$\frac{V}{\sqrt{g R_b' S}} = 3.28 + 5.75 \log \frac{R_b'}{d_{84}} \tag{3}$$

where

g = acceleration due to gravity
S = energy slope
d_{84} = particle size of which 84 percent of the bed is finer

These equations are dimensionally homogeneous and can be applied with any consistent set of units.

Laursen accounted for bed-load transport using a function of the ratio of applied to critical shear stress:

$$\frac{\tau'}{\tau_{ci}} - 1.0 \tag{4}$$

where τ_{ci} is the critical shear stress for size class i. The critical shear stress is obtained from the Shields equation:

$$\tau_{ci} = \theta_{ci}(\gamma_s - \gamma)d_i \tag{5}$$

where

θ_{ci} = critical Shields parameter for size class i
γs = specific weight of sediment
γ = specific weight of water
d_i = mean diameter of size class i

Laursen assumed θ_{ci} had a constant value of 0.039. However, Paintal (1971) determined that the critical Shields parameter varied with applied shear stress. When the dimensionless shear stress for a size class i, θ_i, was less than 0.05, he found that the critical shear stress decreased significantly. Dimensionless shear stress is determined from the following equation:

$$\theta_i = \frac{\tau}{(\gamma_s - \gamma)\,d_i} \tag{6}$$

where τ is the total applied shear stress. This variation in critical shear stress is accounted for in the Laursen-Copeland function by varying the critical Shields parameter between 0.039 and 0.020 when the dimensionless shear stress is less than 0.05. The higher value, recommended by Laursen (1958), was used when θ_i was greater than 0.05. The lower limit was determined by Andrews (1983). The effect of this change is that initiation of motion for coarser particles occurs at lower shear stresses, and the transport potential of coarser particles is increased.

Laursen (1958) used a function of the ratio of total shear velocity $u*$ to the particle fall velocity ω_i to account for suspended sediment transport. In the Laursen-Copeland function, this ratio was modified by including sand and gravel data from both flumes and rivers and by replacing total shear velocity

with grain shear velocity in the functional relationship. A new functional relationship, based on data from both rivers and flumes, was developed for the Laursen-Copeland function. The functional relationship and data scatter are shown in Plate 7. Flume data gathered under more controlled conditions have significantly less scatter than the river data.

Sediment transport is calculated using the following formula:

$$C_s = 0.01 \, \gamma \sum_{i=1}^{N} f_i \left(\frac{d_i}{D}\right)^{\frac{7}{6}} \left[\left(\frac{\tau'}{\tau_{ci}}\right) - 1\right] f\left(\frac{u_*'}{\omega_i}\right) \tag{7}$$

where

C_s = concentration in weight per unit volume
N = number of grain sizes
f_i = fraction of grain size class i in the bed
u_*' = grain shear velocity
$f(u_*'/\omega_i)$ = function defined in Plate 7

This function is considered to be a refinement to Laursen's original equation and is based on a wider range of physical data. The primary benefit is that it moves coarser sediments better than other functions. The sediment-transport function is very sensitive to grain size and to the fraction of fine-sand size classes in the bed. Very large sediment transport capacities may be calculated using this equation if the fraction of fine sands in the bed is not representative of equilibrium flow conditions. This problem is typical of "stand-alone" sediment transport calculations using independently determined hydraulic and sediment variables. The problem is not as prevalent in a numerical model application, because the hydraulic and sediment variables become dependent on each other, and the model's sorting and armoring algorithm tends to adjust the sand fraction on the bed with the sediment inflow.

3 Model Adjustment

Adjustment to 1978-1987 Surveys

The numerical model of the bypass channel was adjusted to simulate the measured accumulated aggradation between 1978 and 1987 surveys. The primary adjustment parameter was sediment inflow. Average sediment transport capacity for a range of discharges was determined for the five upstream cross sections in the bypass channel, and then inflow was adjusted by a constant percentage until the total simulated accumulation of sediment in the bypass channel was the same as the measured accumulation. Sediment inflow rating curves for Stony Fork were the same as at the upstream boundary. Three adjusted numerical models were developed. The first consisted of a single grain size (very fine sand), and sediment transport was calculated using the Laursen-Copeland function. The next two adjusted models developed were multiple-grain-size models which simulated sediment sizes between 0.004 and 256 mm. One of these models used the Laursen-Copeland function to calculate sediment transport, and the other used the combined Toffaleti and Meyer-Peter and Müller functions. Comparisons of calculated and measured longitudinal cumulative aggradation are shown in Figure 4.

Measured sediment inflow data for Bennetts and Stony Fork were limited, with no data for discharges greater than 1,000 cfs. However, the existing data are useful for checking the reasonableness of the adjusted sediment inflow curves. The adjusted total sand inflow curves for the three numerical models are compared to sampled data in Plate 8. A similar comparison for total sediment inflow is shown in Plate 9.

The advantage of the multiple-grain-size models was that the longitudinal distribution of sediment deposition was closer to that measured in the prototype. In addition, the adjusted sediment inflow curves were closer to the measured data. However, the calculated gradation of deposited material was much coarser than the prototype material found in the benches along the low-flow channel. Appropriate quantities of deposited fines in the benches could not be obtained with the multiple-grain-size models using sediment inflow concentrations extrapolated from the measured data.
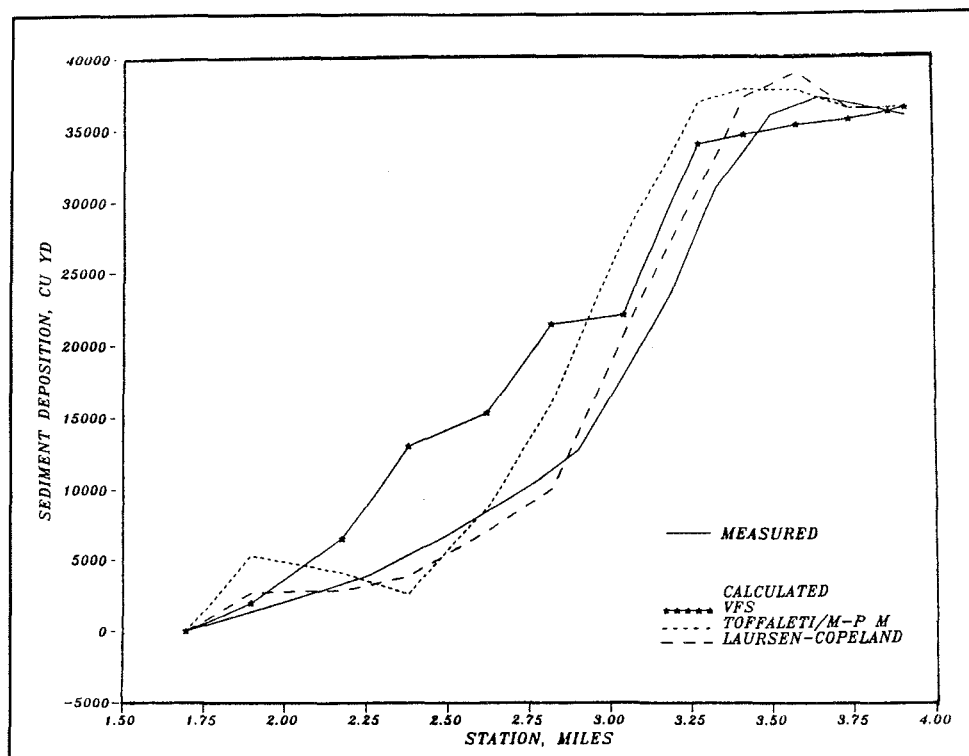
14

Figure 4. Calculated and measured cumulative aggradation in bypass channel, 1978 to 1987

The advantage of the single-grain-size model was that the deposited material was composed of fine material just like that of the benches in the prototype. However, the adjusted sediment inflow at low flow was considerably higher than measured values, and the longitudinal distribution of the deposit was not as good as with the multiple-grain-size models.

The TABS-1 numerical model uses average hydraulic parameters to calculate sediment erosion, transport, and deposition. It does not account for lateral variation in hydraulic conditions and sedimentation processes. Based on bed samples and field observations, it is concluded that there are two distinct sedimentation process occurring in the channel. Coarse sand and gravel are deposited at the upstream end of the bypass channel and at the confluence with Stony Fork. This occurs primarily at high discharges when the larger sized material can be moved by the flow. Deposition of fine sand and silt also occurs in the channel in a bench adjacent to the low-flow channel. The hydraulic conditions that precipitate this bench deposit are uncertain. Flow separation and eddy development at high flow could be responsible, or deposition could occur on the recession of the flood hydrograph when flow depths and transport capacity on the bench become significantly less than in the low-flow channel.

Circumstantiation of Upstream Model

The Yellow Creek bypass channel was resurveyed in January 1992. Between 1987 and 1992, a net accumulation of about 22,400 cu yd was calculated using the average end area method and 28 cross sections. About 11,400 cu yd were eroded from the channel between the concrete chute at mile 1.70 and mile 2.1, while between miles 2.1 and 3.89, about 33,800 cu yd were deposited. Measured accumulated aggradation upstream from the concrete chute is shown in Figure 5.
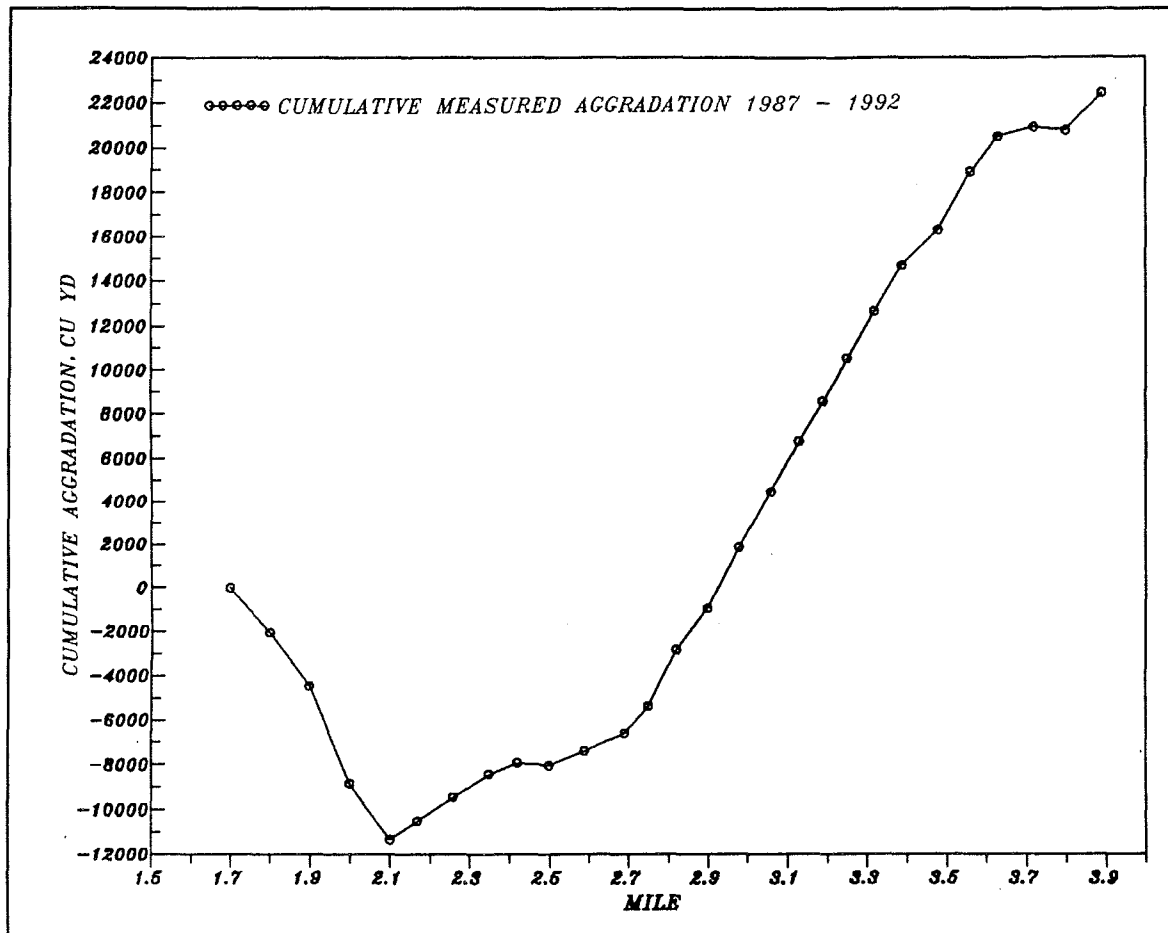

Figure 5. Measured accumulated aggradation in bypass channel between 1987 and 1992

The predictive capability of the TABS-1 numerical model of the bypass channel was tested using the results from the new survey. The historical hydrograph was extended from September 1987 to September 1991 using USGS mean daily flow data from the Yellow Creek near Middlesboro gage. Five peak flows above 3,000 cfs occurred through September 1990, and these data were used to adjust mean daily flow discharges. No adjustment was made to discharges for water year 1991. The total historical simulation totaled 13.75 years.

16

The numerical model was tested using two different initial geometries. The 1987 surveyed cross sections were used with the 1987-1991 hydrograph. The 1978 surveyed cross sections, adjusted to include a low-flow channel, were tested with the 1978-1991 hydrograph. The numerical model did not predict the 1987-1992 prototype degradation that occurred between miles 1.7 and 2.1. This discrepancy is attributed to differences in bed material downstream from mile 2.1. When bed samples were collected in March 1989, no samples were obtained from this reach because water depths were too great for wading and a boat was not available. It is expected that this reach contains significantly finer bed material than the average determined from upstream samples. High runoff events could be responsible for the removal of fine material in the downstream reaches before the 1992 survey. In any event, the numerical model is not considered verified for predicting the behavior of the bed between the concrete chute and mile 2.1.

The numerical model was very successful in reproducing measured aggradation upstream from mile 2.1, as shown in Figure 6. The numerical simulation using 1987 cross sections for initial conditions reproduced 99 percent of the measured deposition. The calculated deposition between 1987 and 1991 from the numerical simulation using the 1978 cross sections for initial conditions reproduced 102 percent of the measured deposition. These results are remarkably consistent for sedimentation studies, and the numerical model is considered circumstantiated for predicting deposition upstream from mile 2.1.

Adjustment to Equilibrium Transport

The existing Yellow Creek channel downstream from the bypass channel is considered relatively stable, and the downstream numerical model was adjusted to obtain a channel with minimum calculated bed change. Several model adjustments were found to have an insignificant effect on results. These included varying the initial bed material gradations in the pool and riffle sections, varying the sediment inflow concentration from Little Yellow Creek, adjusting initial bed elevations at pools, contracting cross sections at pools, and varying roughness coefficients with depth. Other adjustments were found to be significant and were incorporated into the model. Adjustments were made to the initial bed material gradation downstream from mile 12.13 where the model initially calculated excessive degradation. In this reach the initial bed material gradation was coarsened based on calculated bed gradations at the end of several days of high flows. Cross sections upstream from bypass channel mile 0.49 were assigned nonerodible beds because of excessive calculated scour that was not apparent in the prototype. This assignment is reasonable due to the presence of shale bedrock very near the thalweg elevation at mile 0.49, as shown in Plate 10, which is a cross section of the soil stratum based on 1937 borings. Nonerodible beds were also assigned to cross sections between miles 11.37 and 11.85.
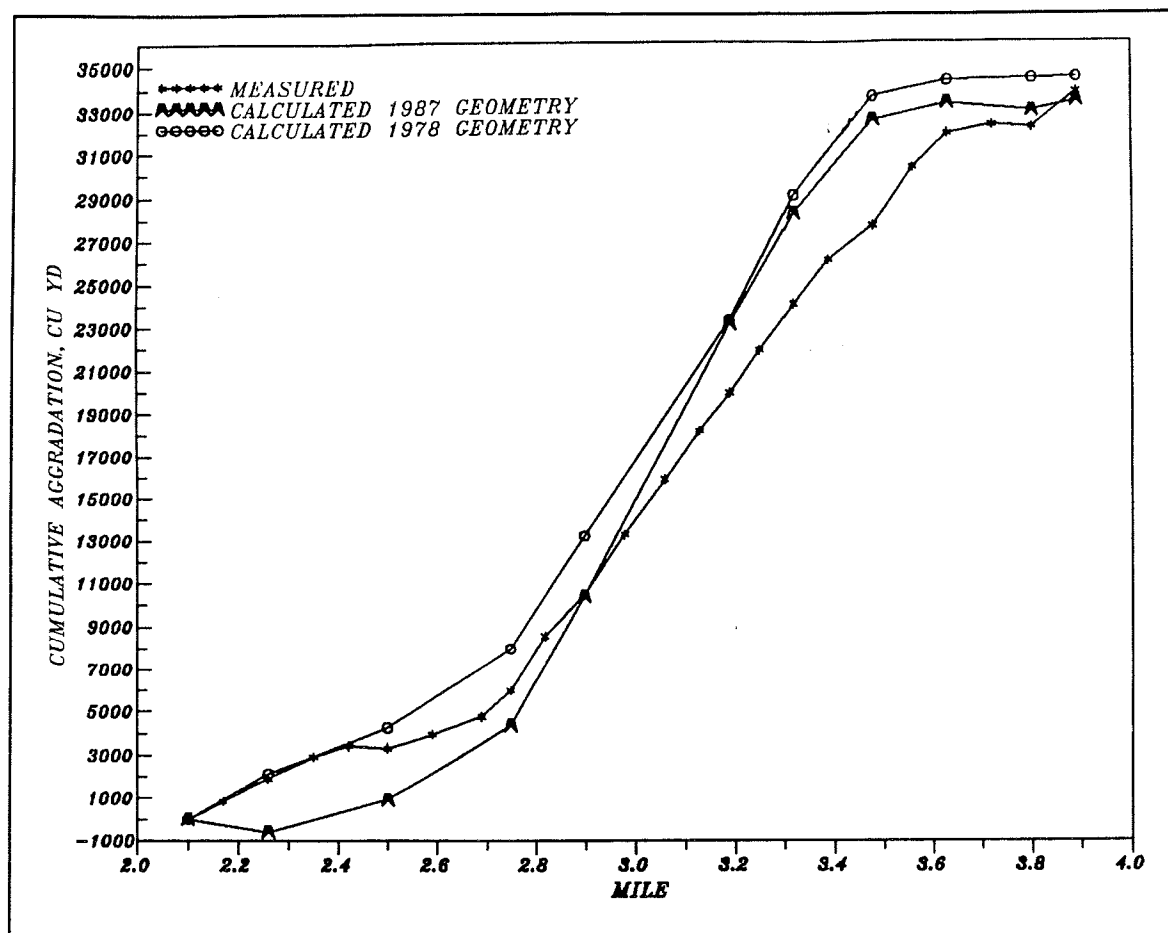
Figure 6. Calculated and measured accumulated aggradation in bypass channel upstream from mile 2.1 between 1987 and 1992

It was determined during the model adjustment that an important streambed control exists between miles 11.37 and 11.85. Surveyed sections in this vicinity indicate a general rise in the normal streambed profile. This could be due to a bedrock outcrop, a resistant bed layer, or a major gravel bar. Initial model runs showed that cross sections in this reach had potential for considerable degradation, which would affect water-surface elevations upstream. In order to simulate measured high-water elevations, these cross sections were given nonerodible bottoms in the numerical model, which in turn resulted in an increase in calculated deposition upstream. The nature of the streambed at this location should be investigated thoroughly during the design of the improved channel, which calls for excavation of about 3.0 ft at mile 11.85. The sensitivity of the model to the bed elevation at this location was tested by replacing the nonerodible bed with an erodible bed thickness of 2 ft. Only about 1 ft of degradation was calculated during the 10-year simulation. Upstream aggradation was reduced by about 0.3 percent.

The calculated thalweg profiles with the adjusted model of the existing channel at the end of the 1978-1987 simulation are shown in Figure 7. The
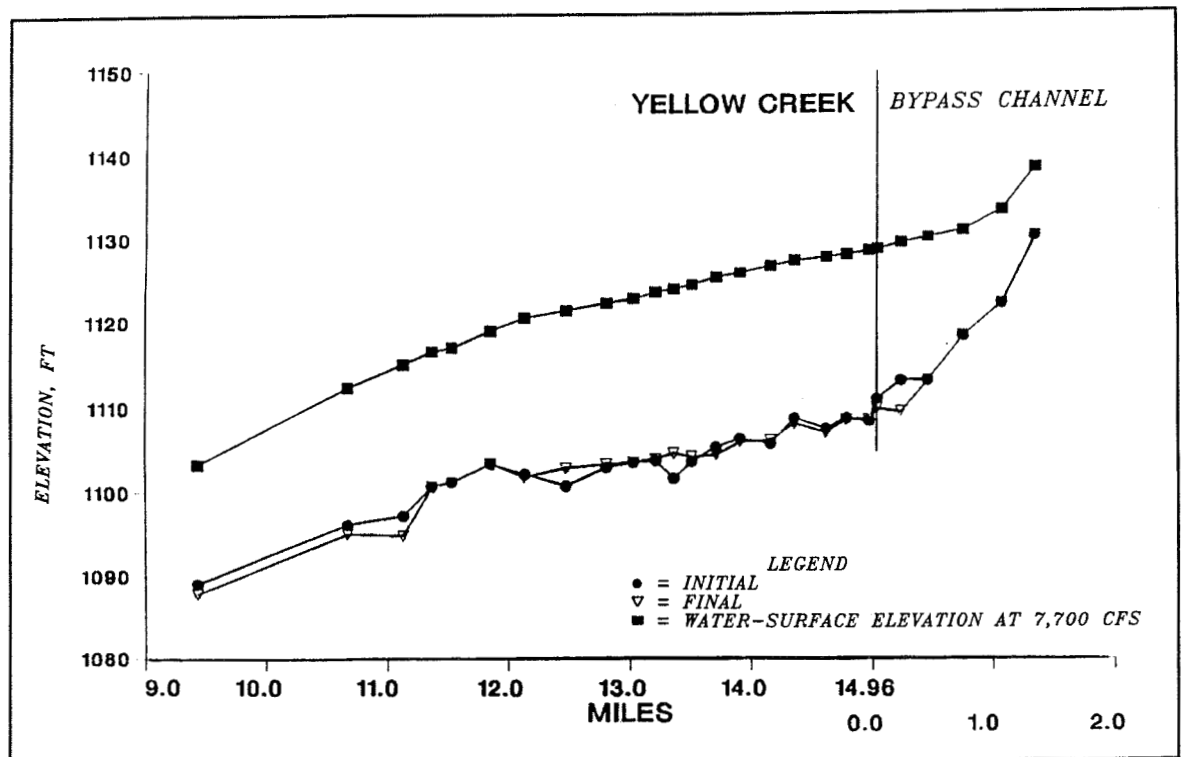
18

Figure 7. Calculated bed elevation change in existing channel after 1978-1987 simulation

model calculated sediment accumulation in the pools upstream from the control at mile 11.85. This accumulation was relatively rapid during the first 2 years of the simulation, but the accumulation rate decreased during the next 3 years. As the simulation proceeded, the model adjusted to its incoming sediment load. The model was essentially stable after about 8,000 cu yd had deposited. Calculated accumulated deposition in the reach between miles 11.53 and 14.95 is tabulated in Table 1.

4   Study Results

The problem of determining the most efficient cross-section shape and an appropriate maintenance dredging strategy for the Yellow Creek bypass channel was addressed using the hydraulic design package for flood control channels, SAM, and the TABS-1 sedimentation model. SAM was used to determine an average cross-section shape that would transport the most sediment and thereby reduce dredging requirements. This required determining a channel-forming discharge and then evaluating transport capacity for various channel cross sections. TABS-1 was used to calculate 10-year deposition for a 1987 base condition and for two dredging alternatives.

Channel-Forming Discharge

Channel-forming discharge is based on the concept that the river channel adjusts itself to the imposed conditions of inflowing water and sediment. In this study channel-forming discharge is defined as the discharge increment that transports the most sediment. This increment can be determined by step-wise integration of the flow duration and sediment rating curves. The channel-forming discharge for the Yellow Creek bypass channel was calculated at the concrete chute and upstream of the confluence with Stony Fork.

The USGS provided a flow duration curve, based on 46 years of record, for the Yellow Creek at Middlesboro gage, located at mile 11.4. This curve was adjusted using mean daily flow records for application to the bypass channel. Mean daily flow at the concrete chute was determined by adding mean daily flow records from gages on Stony and Bennetts Forks, the sum of which, on the average, was about 50 percent of the mean daily flow at the Yellow Creek gage. Upstream from Stony Fork, mean daily flows were, on the average, about 25 percent of those on Yellow Creek. Flow duration curves are shown in Plate 11. It should be noted that mean daily flow percentages are different from peak discharge percentages. Statistical 100-year-frequency peak discharges and unit hydrograph discharges for the Standard Project Flood (SPF) are compared in the following tabulation.

Sediment discharge rating curves were developed from data collected from Stony Fork and Bennetts Fork between 1985 and 1988. These data were

20

Location	Percentages of Yellow Creek Discharge in Bypass Channel		
	SPF	100-year	Mean Daily
Upstream from Stony Fork	51	67	25
Concrete Chute	92	117	50

limited, and extrapolation was required for discharges above 500 cfs. Both
total sediment loads and total sand loads were used in the step-wise integration
to determine channel-forming discharge.

The incremental integration of the flow duration and sediment transport
curves was accomplished using SAM. Results, shown in Figures 8 and 9,
indicate that the maximum sediment load is transported in the discharge
interval between 100 and 200 cfs upstream from Stony Fork and between 200
and 400 cfs at the concrete chute. These discharges are exceeded between 2
and 6 percent of the time in the bypass channel. These relatively low values
for the discharge increment carrying the maximum sediment load are attributed
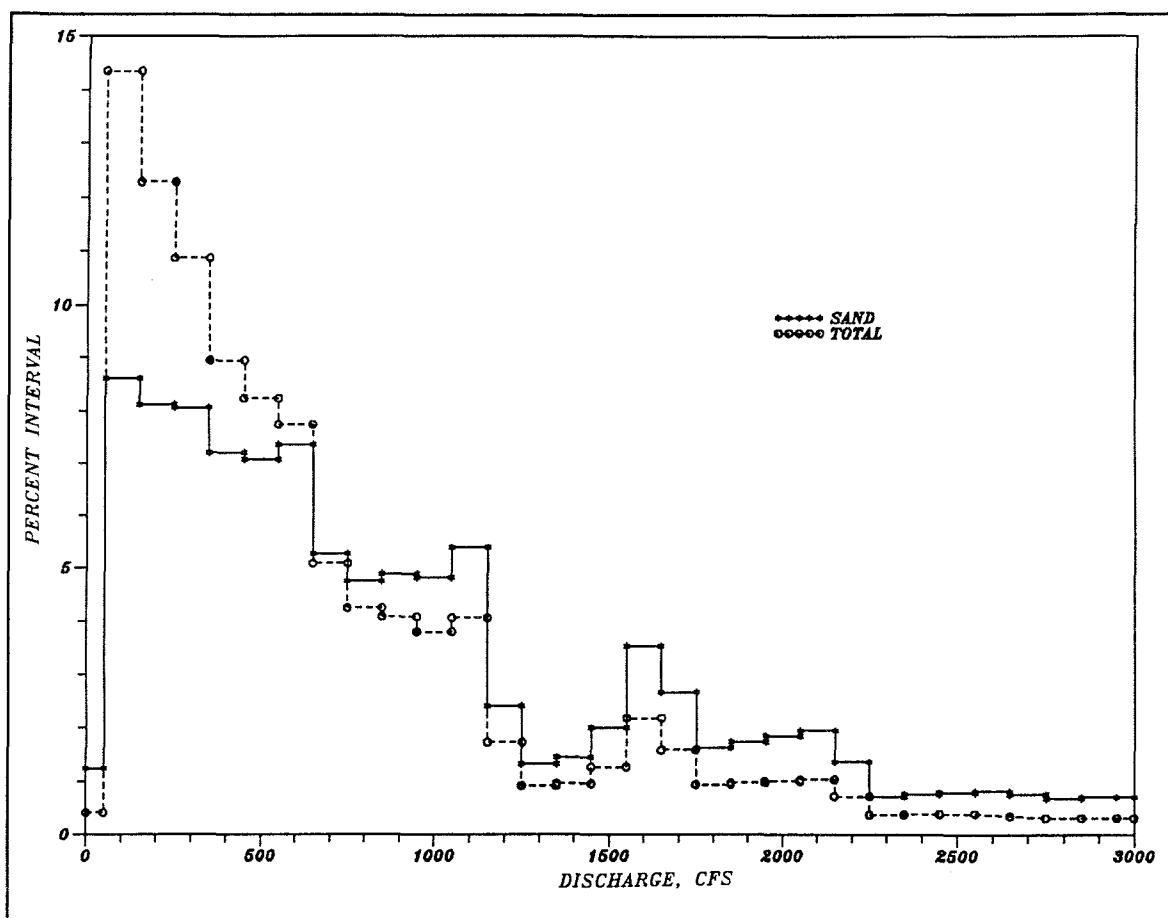

Figure 8. Sediment yield by discharge interval for bypass channel upstream from Stony Fork
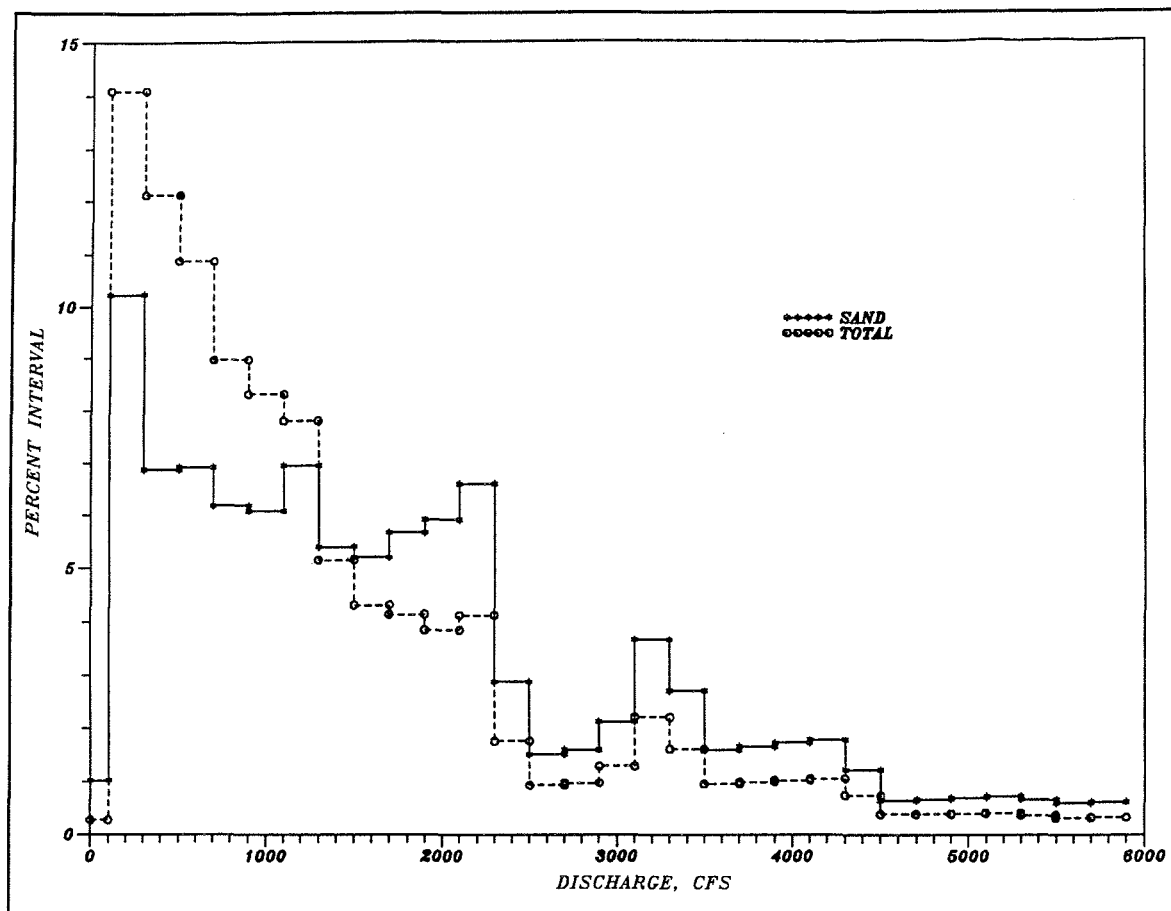
Figure 9. Sediment yield by discharge interval for bypass channel at chute

to the very high suspended fine load in the bypass channel. The high fine load may be supplied by the heavy mining activity in the watershed. About two-thirds of the total sediment load is transported by discharges less than 800 cfs upstream from Stony Fork and by discharges less than 1,600 cfs at the concrete chute. However, only 53 percent of the sand load was transported at discharges less than 700 cfs upstream from Stony Fork and 50 percent at discharges less than 1,600 cfs at the concrete chute. Channel-forming discharges of 200 and 400 cfs were assigned for the bypass channel upstream from Stony Fork and at the concrete chute, respectively.

Composite Section Design

A composite section with a low-flow channel is designed to provide more efficient hydraulic conditions for sediment transport at low flow without sacrificing hydraulic efficiency at high flows. The SAM program was used to compare transport efficiency for composite sections with 2-, 4-, and 6-ft-deep low-flow channels. Base widths for the low-flow channel were determined by

22

averaging the base widths of existing cross sections upstream and downstream from the confluence with Stony Fork, which were 25 ft and 40 ft, respectively. Side slopes of 1V:2H were assigned. Of course, other geometries may be appropriate, but using existing base widths for the low-flow channel is consistent with the geomorphic principle of the stream being the best model of itself. Transport capacity in a single trapezoidal channel with a 120-ft base width was also calculated for comparison.

The average energy slopes for the two reaches in the bypass channel were determined using the HEC-2 backwater model prepared from 1978 geometry. Average energy slopes were determined for the bypass channel between the concrete chute and Stony Fork and between Stony Fork and mile 3.39. In general, energy slope decreased with discharge downstream from the confluence with Stony Fork and increased with discharge upstream from the confluence with Stony Fork.

Hydraulic properties in the composite cross section were determined separately for both the low-flow channel and the bench. Sediment transport was calculated from these hydraulic parameters using the Meyer-Peter/Müller equation, which is appropriate for gravel-bed streams, and the Laursen-Copeland and Toffaleti and Meyer-Peter/Müller equations, which are appropriate for combination sand and gravel-bed streams. Results are compared in Plates 12-17.

The sediment transport curves were interpreted to choose the most efficient composite cross section. Look first at the curves calculated using the Meyer-Peter/Müller equations upstream from the confluence with Stony Fork (Plate 12). The curves indicate that the most sediment is transported with a 6-ft-deep low-flow channel. Since discharges below 500 cfs are contained within the 4-ft-deep low-flow channel, the transport capacity of the 4-ft and 6-ft low-flow channels are the same below 500 cfs. Since the channel-forming discharge is 200 cfs, the 4-ft-deep and 6-ft-deep low-flow channels would operate with equal efficiency at that discharge. Transport capacity in the 2-ft low-flow channel is less for all discharges between 100 and 3,000 cfs. Capacity in the trapezoidal channel is negligible at discharges less than 1,200 cfs.

Sediment transport capacity curves calculated using the Laursen-Copeland and Toffaleti and Meyer-Peter/Müller equations are similar (Plates 13 and 14). At discharges less than about 1,500 cfs, the 6-ft low-flow channel is the most efficient. At discharges less than about 500 cfs the 4-ft and 6-ft low-flow channels are the same. The 2-ft low-flow channel is the least efficient composite section. The trapezoidal cross section is more efficient in terms of the fine load and its efficiency increases with discharge; however, it does not exceed the efficiency of the 4-ft low-flow channel for discharges less than 1,000 cfs, or the 6-ft low-flow channel for discharges less than about 1,500 cfs. For a dominant discharge of 200 cfs the 4-ft composite is sufficient.

In the bypass channel downstream from Stony Fork, the sediment transport rating curves also suggest that the 6-ft-deep low-flow channel is the most efficient. However, at the channel-forming discharge of 400 cfs, the 4-ft-deep low-flow channel is equally efficient. The curves for the Meyer-Peter/Müller equation (Plate 15) indicate that there is no transport of sediment at the channel-forming discharge, but the Laursen-Copeland (Plate 16) and combined Toffaleti and Meyer-Peter/Müller equations (Plate 17) indicate essentially equal transport capacity at the channel-forming discharge for both the 6-ft and 4-ft-deep low-flow channels.

Another more detailed method for assessing the transport capacity of the channel is to integrate the flow duration curve with the transport curves. Sediment yield based on capacity of the cross sections for the reach upstream of Stony Fork was calculated using the Laursen-Copeland function as listed in the following tabulation:

Section	Percent of Total Yield with Trapezoidal Section
2-ft low-flow channel	136
4-ft low-flow channel	153
6-ft low-flow channel	155

The small increase gained using the 6-ft low-flow channel does not justify extra excavation cost, and the 4-ft low-flow channel is recommended for this reach. A similar analysis was conducted for the reach downstream from the confluence with Stony Fork as listed in the following tabulation:

Section	Percent of Total Yield with Trapezoidal Section
2-ft low-flow channel	101
4-ft low-flow channel	118
6-ft low-flow channel	124

The 4-ft low-flow channel is also recommended for this reach.

Dredging Alternatives

The adjusted and circumstantiated TABS-1 numerical model of the bypass channel was used to evaluate dredging alternatives. Initial channel geometry, related to the depth and width of the low-flow channel, was used as one of the adjustment parameters. Typically, channel geometry does not significantly

24

affect calculated results. However, at certain discharges, the composite cross section in the Yellow Creek bypass channel induces significantly different hydraulic conditions on the channel bench and in the low-flow channel. When this occurs, the average hydraulic parameter assumption in the one-dimensional model is not representative of prototype conditions. Since deposition in the bypass channel was found to be sensitive to the initial channel geometry, reliable quantitative results cannot be obtained from the numerical model when evaluating various channel geometries. Another way of stating this dilemma is that the model cannot be used to evaluate a parameter that was also an adjustment parameter. This effect was minimized by retaining the same initial cross-section shape in the dredging evaluations. Different dredging schemes were identified by raising or lowering the cross-section elevation.

Dredging alternatives were evaluated by calculating deposition in the bypass channel using the 1978-1987 hydrograph. Calculated depositions were compared for the same 10-year hydrograph using the existing (1987) channel geometry as a base test. Dredged cross sections for the alternatives were assigned a composite geometry with a 4-ft-deep low-flow channel 25 ft wide upstream from Stony Fork and 40 ft wide downstream from Stony Fork. Dredging occurred between miles 2.75 and 3.48. The first alternative was to dredge about 26,000 cu yd to about the same average elevation as the 1978 dredging operation. The second alternative was to dredge about 44,100 cu yd to an average elevation about 1 ft lower than the 1978 dredging operation.

Aggradation in the bypass channel was calculated using the multiple-grain-size Laursen-Copeland transport function. The progression of total accumulated sediment in the bypass channel for the 14-year simulation is shown in Figure 10. In this figure, initial dredging volumes are indicated on the ordinate. Average accumulation rates in the bypass channel based on the 1978-1991 hydrograph and percent increase in dredging with the dredging alternatives are shown in the following tabulation:

| Alternative | Total Calculated cu yd | Percent of Base Test | Accumulation Rate cu yd/year | | |
			3 years	10 years	14 years
1987 Geometry	78,300	100	5,320	3,980	5,600
Alternative 1	86,400	110	8,180	5,140	6,170
Alternative 2	95,500	122	9,230	5,780	6,820

Sensitivity to Transport Function

The Toffaleti and Meyer-Peter/Müller multiple-grain-size transport function was used to evaluate model sensitivity to transport function and sediment inflow. Sediment inflow for the Toffaleti and Meyer-Peter/Müller equation
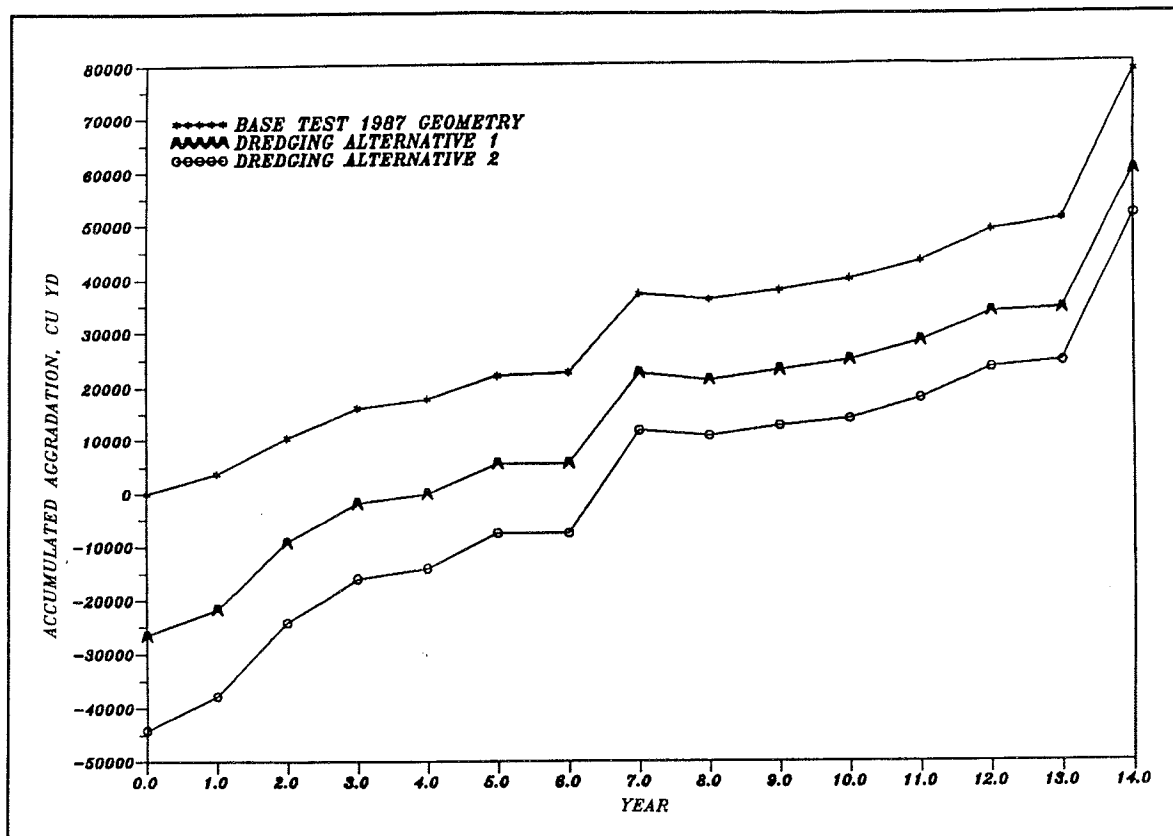
Figure 10. 14-year progression of accumulated sediment deposition in bypass channel for dredging alternatives using Laursen-Copeland function

was determined in the same manner as with the Laursen-Copeland function: by adjusting the sediment inflow until the 1978-1987 aggradation simulated measured aggradation using 1978 geometry with an imposed low-flow channel for initial conditions. In the evaluation of dredging alternatives, the same channel geometry and hydrographs were used with the two multiple-grain-size sediment transport functions. Average accumulation rates in the bypass channel based on the 1978-1991 hydrograph are shown in the following tabulation:

Alternative	Total Calculated cu yd	Percent of Base Test	Accumulation Rate cu yd/year		
			3 years	10 years	14 years
1987 Geometry	56,800	100	5,590	3,430	4,058
Alternative 1	73,100	129	6,144	4,150	5,221
Alternative 2	73,400	129	6,845	4,420	5,246

Variations between the results of this test and the test using the Laursen-Copeland function are attributed to differences in sediment inflow and transport function, and how the two functions respond to changes in channel geometry. This comparison provides a confidence interval for the numerical results.

The Meyer-Peter/Müller equation was used as a single-grain-size transport function to evaluate the effect of the dredging alternatives on bed load. Fine gravel (5.6 mm) was used as the representative grain size in the model. Model results showed that about 97 percent of the inflowing bed load was trapped in the bypass channel regardless of the initial geometry. For all cases tested, most of the deposition was immediately downstream from the confluence with Stony Fork and between the confluence of Stony Fork and mile 3.63. This test showed that bed load accumulation is not significantly influenced by the dredging alternatives tested.

Finally the single-grain-size model using very fine sand and the Laursen-Copeland function was tested. When the progressive accumulation of sediment for the 10-year hydrograph was plotted, it was apparent that calculated results showed no progression at all, but rather an annual oscillation. The model showed deposition in low-flow years and degradation in high-flow years. This result is contrary to known conditions where flood years bring the greatest accumulation of sediment. Due to this inconsistency with prototype behavior, the fine-grain single-size analysis was discontinued.

Long-Term Effects

The long-term effects of dredging alternatives were evaluated by calculating deposition in the bypass channel using the 1978-1991 hydrograph repeated four times for a 55-year simulation.

Aggradation in the bypass channel was calculated using the multiple-grain-size Laursen-Copeland transport function. The progression of total accumulated deposition in the bypass channel for the 55-year simulation is shown in Figure 11. In this figure, initial dredging volumes are indicated on the ordinate. Accumulation rates and percent increase in dredging with the dredging alternatives are shown in the following tabulation. Accumulation rates shown are cumulative for the number of years indicated.

Sediment deposition for the 55-year simulation significantly altered cross-section geometry and depleted flood conveyance upstream from mile 2.5. In the numerical model, movable-bed limits had to be extended to allow for a wider depositional area. The significant change in cross-section shape further strained the average hydraulic parameter assumption inherent to the one-dimensional TABS-1 code. As the simulation progressed, the model results became more questionable. The calculated accumulation rates are interesting for comparison, but the 55-year deposition quantities are impractical, unless levee heights are increased.
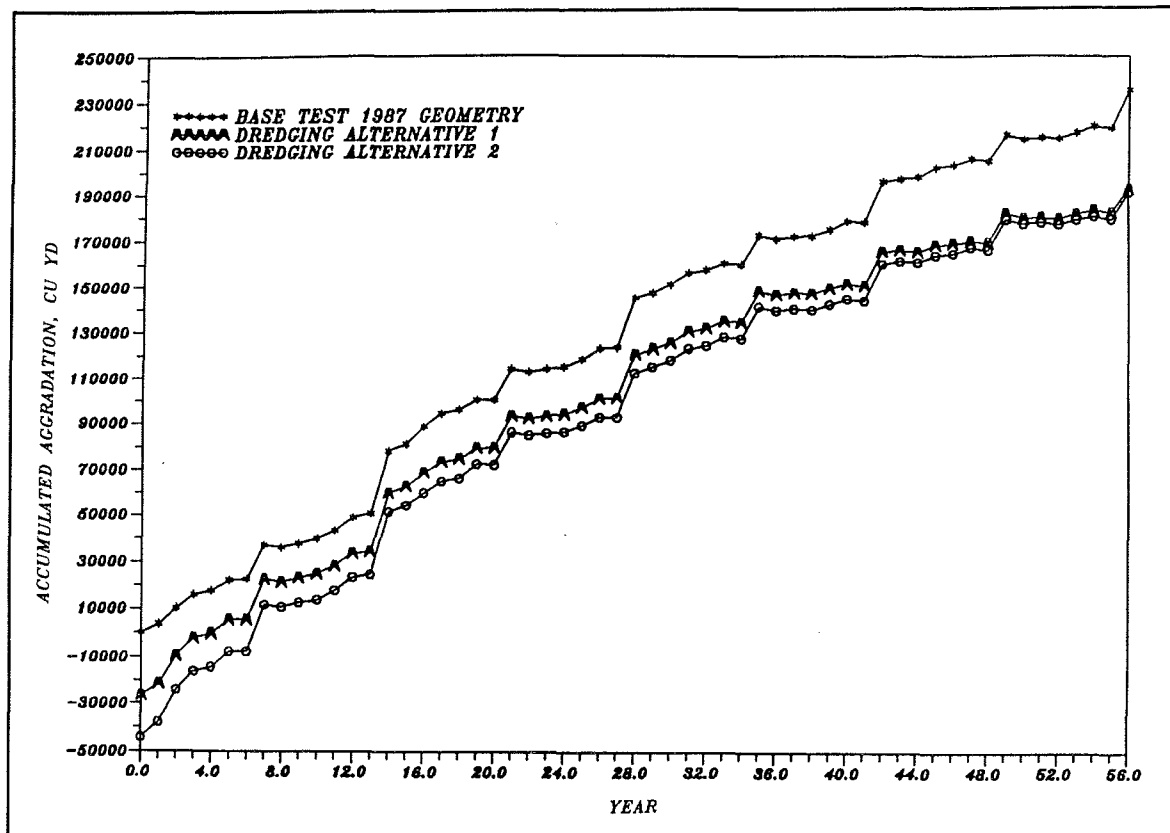
Figure 11. 55-year progression of accumulated sediment deposition in bypass channel for dredging alternatives using Laursen-Copeland function

Alternative	Total Calculated cu yd	Percent of Base Test	Accumulation Rate, cu yd/year for 14-year Increment			
			1st	2nd	3rd	4th
1987 Geometry	235,500	100	5,600	4,820	3,610	2,790
Alternative 1	219,700	93	6,170	4,360	3,260	1,890
Alternative 2	235,500	100	6,820	4,380	3,450	2,170

Using the 1978-1991 hydrograph, it took about 4 years for alternative 1 to fill back to predredging conditions and about 6.5 years for alternative 2. During the first 3 years, accumulation rates were 154 and 173 percent of the undredged channel for alternatives 1 and 2, respectively. However, as the channel filled with sediment, the accumulation rates decreased, and there was less and less distinction between the alternatives. After the first 28 years, differences in accumulation rates may be on the same order or magnitude as the accuracy of the numerical model, and results must be interpreted with care. It may be concluded, however, that even though the rates decrease, the decrease

28

is relatively small, and equilibrium conditions will not be attained in the Yellow Creek bypass channel even after 55 years.

Standard Project Flood

The performance of dredging alternatives during the Standard Project Flood was compared using the numerical model. The Standard Project Flood hydrograph was supplied by the Nashville District. The calculated discharge at the concrete chute was used in the model from the downstream boundary to the confluence with Stony Fork; and the calculated discharge upstream of the Stony Fork confluence was used from that point to the upstream model boundary. The sediment inflow rating curve for the numerical model was extrapolated beyond discharges used to circumstantiate the model. This results in some degree of uncertainty with respect to predicting actual prototype performance and must be considered when interpreting results.

Aggradation in the bypass channel during the Standard Project Flood was calculated using the multiple-grain-size Laursen-Copeland transport function. Total sediment accumulation in the bypass channel at the peak and at the end of the Standard Project Flood are shown in Figures 12 and 13, respectively. In these figures, initial dredging volumes are indicated on the ordinate. Total sediment accumulation volumes for the dredging alternatives are shown in the following tabulation:

	At Peak		At End	
Alternative	Total Calculated cu yd	Percent of Base Test	Total Calculated cu yd	Percent of Base Test
1987 Geometry	71,900	100	112,700	100
Alternative 1	78,400	109	134,700	120
Alternative 2	82,800	115	140,900	125

Calculated water-surface elevations at the Standard Project Flood peak for initial bed conditions and for calculated bed conditions considering sediment deposition are shown in Table 2. The numerical model does not include losses due to bridges, so that these profiles should be used for comparison only.

Assessment of Downstream Design

The proposed Yellow Creek design channel downstream from the bypass channel calls for increasing the channel width to 100 ft through most of the study reach between miles 11.85 and 14.96, and lowering the bed elevation at
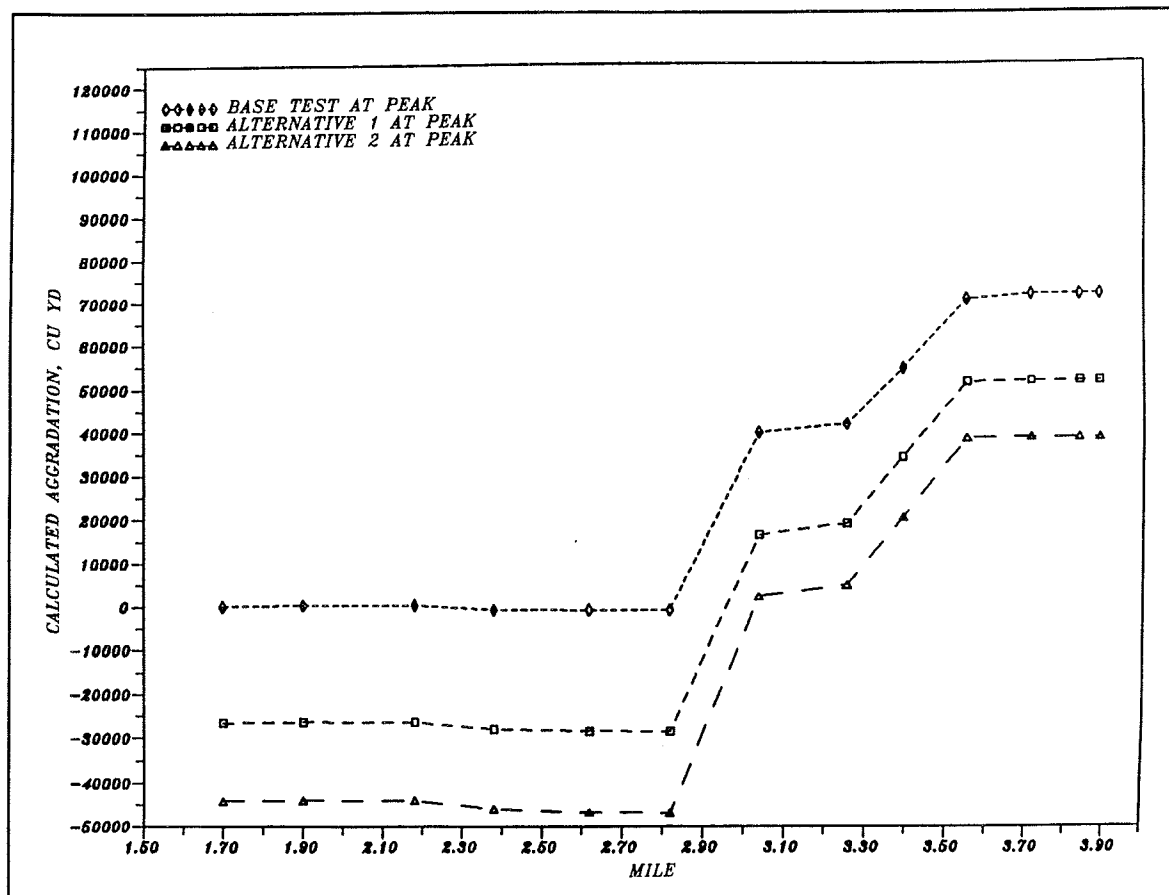
Figure 12. Longitudinal accumulated sediment deposition in bypass channel at peak of Standard Project Flood

mile 11.85 by about 3 ft. The numerical model's geometry was revised to account for these changes and tested with the 1978-1987 hydrograph. Increases in calculated degradation or aggradation from the existing channel were attributed to the design of the proposed channel. During the first year, 34,100 cu yd of aggradation and 3,900 cu yd of degradation were calculated. After 10 years, 44,700 cu yd of aggradation and 11,300 cu yd of degradation were calculated. Calculated net accumulated deposition for the design channel and the sediment deposition attributed to the channel improvements are shown in Table 3 and Figure 14. Comparing the difference in calculated accumulations between the existing and design channels shows that, on the average, about 32,000 cu yd of sediment deposition can be expected in the proposed channel unless an annual maintenance program is implemented. Most of this accumulation would occur during the first year.

Interpreting model results in pool sections is difficult. The numerical model calculates sediment deposition in pools, which are identified by dips in the channel invert profile, in an attempt to produce a uniform bed slope. In both the existing and design channels, the numerical model calculated about the same depth of deposition at pools located at miles 13.35 and 12.47.
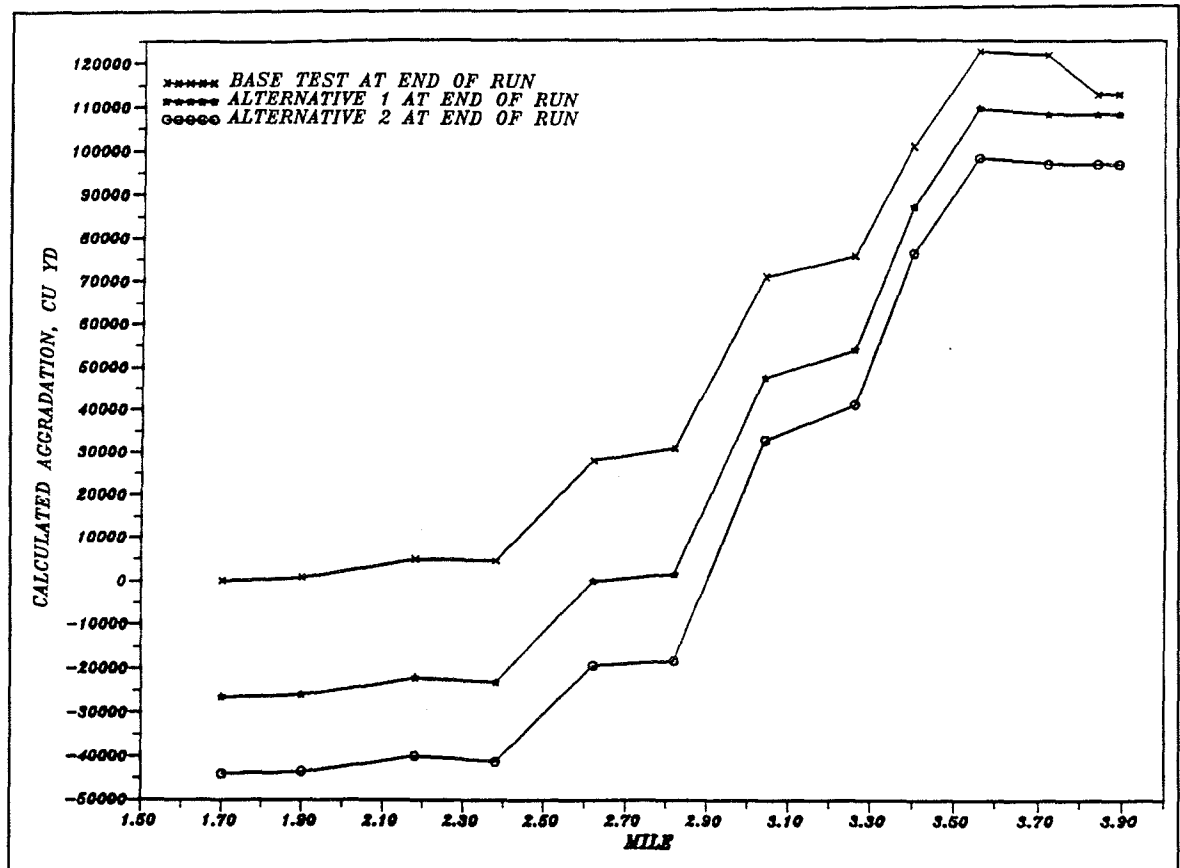
30

Figure 13. Longitudinal accumulated sediment deposition in bypass channel at end of Standard Project Flood

However, a larger volume of sediment deposited in the design channel because it is wider. In the model of the existing channel, deposition in the pools can be attributed to two-dimensional effects not accounted for in the one-dimensional model. These include flow concentrations due to contractions, bends, or obstructions. The design channel will be more streamlined with a constant width in most reaches, so that the presence of two-dimensional effects is less likely. Therefore, deposition in pool cross sections in the design channel cannot be ignored.

In addition to the pools, significant sediment accumulation was calculated in the first 0.5 mile of the improved channel. This accumulation is attributed to the decrease in sediment transport potential with the widened channel so that it is unable to transport all of the incoming sediment load.

The numerical model of the design channel indicated that degradation would occur in reaches where the channel was not widened. Local scour protection should be provided at these locations. Estimates of degradation were made based on differences in calculated bed changes between the existing and design channels. At mile 13.02 the channel was not widened due to constraints by highways on both sides of Yellow Creek, and about 1.1 ft of
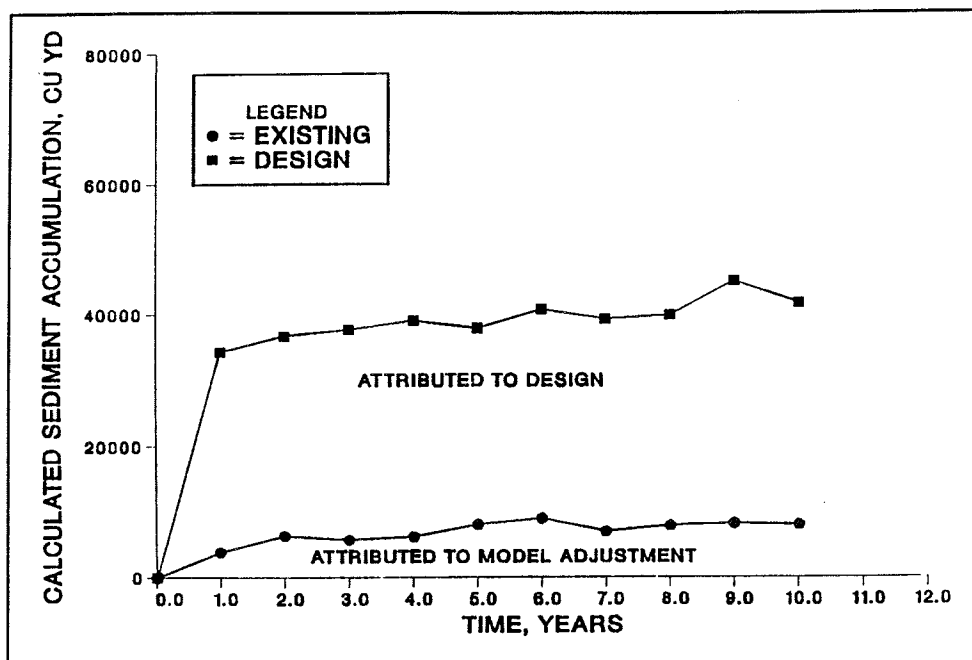
Figure 14. Calculated net sediment accumulation between miles 11.53 and 14.96 for existing and design channels

degradation was calculated. Lowering of the bed at mile 11.85 could also be a contributing factor to the degradation at mile 13.02. Degradation of about 1.6 ft was calculated at the railroad bridge at mile 14.15 where the channel retained its existing width in the proposed design. Degradation of about 3.7 ft occurred in the diversion canal upstream from its confluence with Yellow Creek. This degradation is attributed to lowering of the water-surface elevations in the improved creek channel. This problem could be corrected with a head cut control structure. Otherwise, degradation may be accompanied by lateral migration of the stream, which could threaten existing levees and adjacent property.

The difference in calculated bed change and volume accumulation between the existing and proposed channels is shown in Figure 15. In the pools, bed change differences were small, but due to the wider channel in the proposed design, sediment accumulation was greater.

Adjustment for Deposition

For purposes of calculating design water-surface elevations, design invert elevations should be increased to account for sediment deposition. Deposition depths were determined using the difference in calculated accumulated volumes at each cross section rather than the difference in bed change. Bed elevations can be calculated from the volume differences using the reach lengths and movable-bed widths from the numerical model. The adjusted
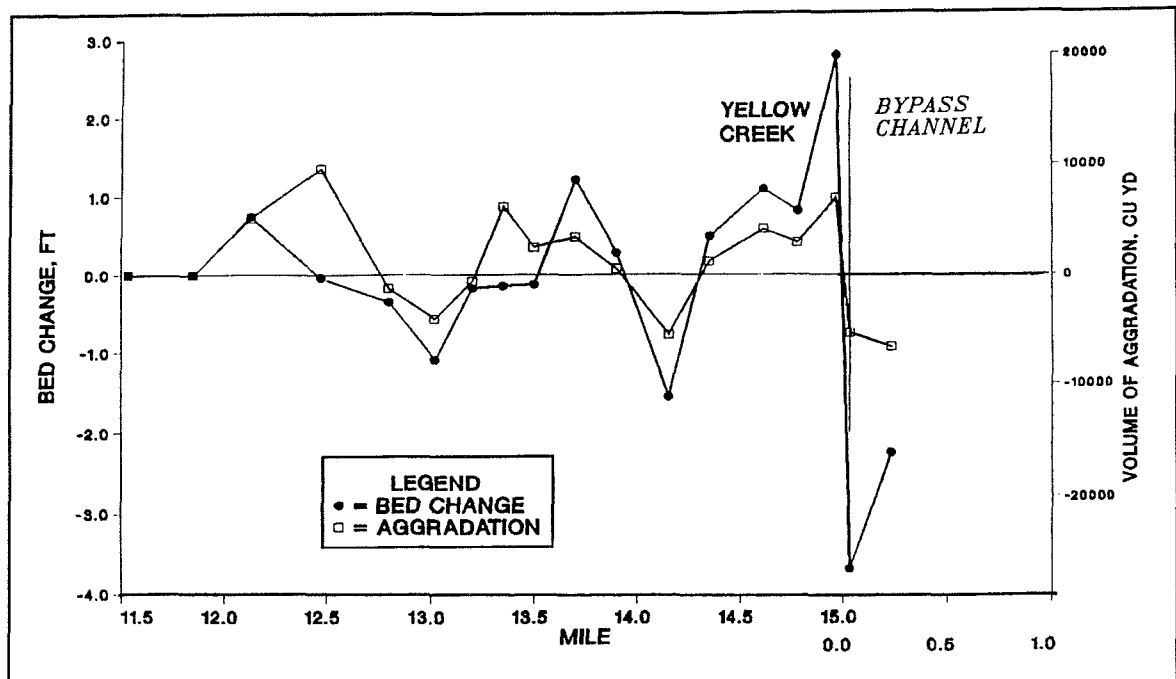
32

Figure 15. Difference in calculated bed change and sediment accumulation, by reach, between existing and design channels after 1978-1987 hydrograph

initial bed elevations were incorporated into the numerical model, which was run with the 10-year historical hydrograph. The results of this test indicated that further increases of bed elevations in the pool sections would be appropriate. A second test with the refined initial bed elevations produced the recommended design invert elevations shown in Table 3. The difference in calculated net accumulated volumes between the existing and design channels with the revised invert elevations after the 10-year simulation was only 2,700 cu yd, compared to 33,400 cu yd with the original bed elevations. This included 14,600 cu yd of aggradation, 60 percent of which occurred in the first 0.5 mile downstream from the diversion canal; and 11,900 cu yd of degradation, 72 percent of which occurred due to constrictions at miles 14.15, 13.02, and 12.80.

Average Annual Deposition

Average annual deposition can be determined using a representative annual hydrograph or a long-term historical hydrograph. This determination is complicated for the design channel due to its initial instability with respect to sediment transport. The numerical model investigation demonstrated that significant sediment deposition will occur initially, decreasing with time. In the first year (1978) of the historical simulation, a total of 34,100 cu yd and a net of 30,200 cu yd deposited in the design channel. This rate of deposition decreased as the simulation proceeded, so that after 10 years, a total of 44,700 cu yd and a net of 33,400 cu yd deposited. Using a 10-year average,

total and net average annual depositions of 4,500 and 3,300 cu yd, respectively, are calculated.

Another way to calculate the average annual deposition is using the flow duration hydrograph. The flow duration hydrograph has the advantage of including the effects of infrequent high discharges; however, its flow distribution is unrealistic in terms of an actual annual runoff season. The flow duration hydrograph is compared to the 1978 hydrograph in Plate 2. Total sediment inflow using the flow duration hydrograph was about 11 percent lower than with the 1978 hydrograph, which indicates that the 1978 hydrograph had slightly more runoff than an average annual hydrograph. Comparing the difference in calculated sediment deposition between the existing and design channels using the flow duration hydrograph, a total of 16,700 cu yd of aggradation and 9,600 cu yd of degradation were calculated in the channel on an annual basis.

The discrepancy in average annual deposition calculations using the different methods is attributed to the importance of intermediate discharges to sediment deposition in Yellow Creek and the initial sediment transport deficiency in the design channel. Under these conditions, it is more appropriate to use a realistic hydrograph. Therefore, average annual deposition calculated using the 1978 hydrograph is adopted for determining annual maintenance estimates for a channel returned to its design condition every year.

Sensitivity Tests in Downstream Channel

The sensitivity of numerical model results to sediment inflow was evaluated. Envelopes of the calculated outflow from the upstream numerical model using the 1978-1987 hydrograph were used to determine high and low sediment inflow rating curves. When sediment inflow was reduced by 50 percent, sediment accumulation in the design channel after the 10-year simulation was reduced about 34 percent. When the sediment inflow was increased by 80 percent, sediment accumulation in the design channel at the end of the 10-year simulation increased by 43 percent. These results demonstrate that sediment deposition in the design channel is sensitive to sediment inflow. The general distribution of sediment is not sensitive to sediment inflow, as shown in Figure 16.

Measured suspended sediment data were used to develop a sediment inflow rating curve that had lower concentrations at low discharges. This curve was based on data from gages on Yellow Creek, Bennetts Fork, and Stony Fork, taken at primarily low discharges. Most of the data are for total sediment concentration, and just a few include a particle size analysis that identifies the sand load. The sand inflow curve was determined optically from the plotted data up to 1,000 cfs. Above this discharge, the calculated sediment inflow was used. The sediment inflow rating curves are compared in Plate 18. The revised sediment inflow curve was incorporated into the numerical model and
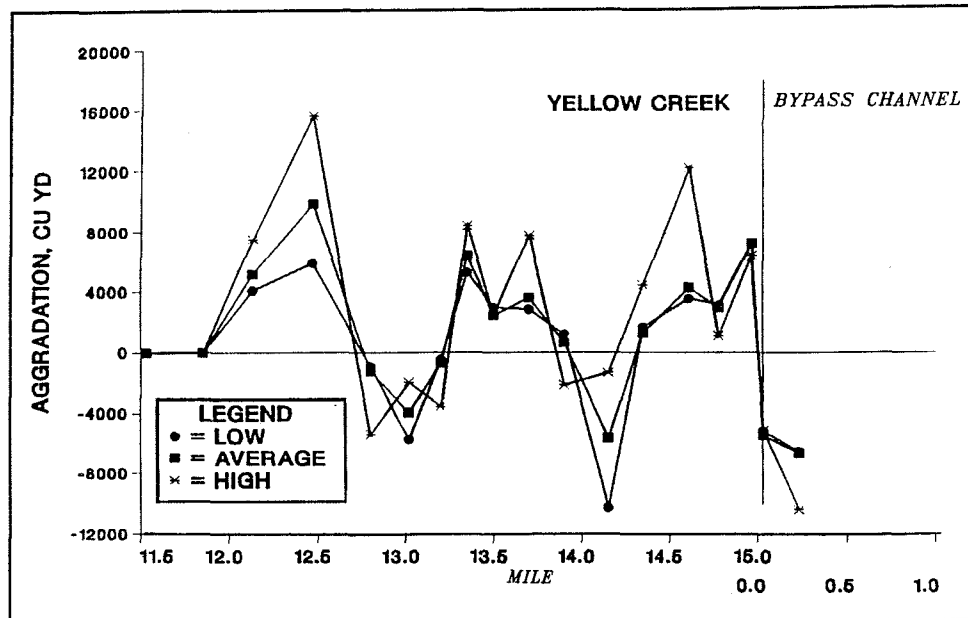
Figure 16. Difference in calculated sediment accumulation, by reach, between existing and design channels after 1978-1987 hydrograph, testing model sensitivity to sediment inflow

run with the design and existing channel, and was found to have an insignificant effect on study results. The accumulated sediment deposition in the design channel with the calculated and measured sediment inflow curves after the 10-year simulation is shown in Plate 19.

The numerical model was used to test the effect of decreasing the design roughness coefficients in the improved channel. In the original design, the roughness coefficients for the design channel were assigned the same value as for the natural channel. If the bank roughness is significantly higher than the bed roughness, then widening the channel should reduce the composite channel roughness coefficient. Using the Limerinos equation and a D_{84} of 10 mm a bed roughness of 0.022 was determined. A bank roughness of 0.050 was then calculated assuming a composite roughness of 0.038 in the existing channel. Using these values, a composite roughness coefficient of 0.030 was calculated for the improved channel. This value was incorporated into the numerical model to test the effect of the possible lower design roughness coefficients. At the end of the 10-year simulation, accumulated aggradation attributed to the design channel with the original invert elevations was about 10 percent or 3,200 cu yd less with the lower roughness coefficients. An additional 1.1 ft of scour occurred in the diversion channel upstream from the confluence with Yellow Creek.

5 Conclusions and Recommendations

Conclusions

The TABS-1 numerical model was adjusted to simulate measured deposition between 1978 and 1987 in the Yellow Creek bypass channel. The adjusted model was circumstantiated by successfully simulating aggradation between 1987 and 1991 in the bypass channel upstream from mile 2.1, which was the reach of primary interest in terms of dredging. The numerical model could then be used to evaluated dredging alternatives in the bypass channel upstream from mile 2.1.

The numerical model of Yellow Creek between miles 9.43 and 14.96 and the bypass channel between miles 0.0 and 1.38 was adjusted to obtain minimum calculated bed change over a 14-year period. The adjusted model of existing conditions stabilized after about 8,000 cu yd had deposited. This quantity was subtracted from calculated deposition in the proposed enlarged channel to obtain projected deposition quantities.

The TABS-1 model was used to evaluate two dredging alternatives in the bypass channel. Alternative 1 called for 26,000 cu yd of dredging, and alternative 2 called for 44,100 cu yd. Using the 1978-1991 hydrograph, alternative 1 took 4 years to aggrade back to predredging conditions, and alternative 2 took about 6 years. During the first 3 years, accumulation rates were 154 and 174 percent of the undredged channel for alternatives 1 and 2, respectively. Thus, both initial and maintenance dredging quantities would be greater with alternative 2, but dredging would be required less frequently. An economic analysis would be required to determine which alternative was the most cost effective.

Deposition rates for both dredging alternatives and the existing channel generally declined with time; but even after 55 years of simulation, an equilibrium condition, where inflowing sediment could be transported through the reach, was not achieved.

Sediment deposition in the bypass channel prior to the peak of the Standard Project Flood had a significant effect on calculated water-surface elevations. The sediment deposition resulted in a maximum difference in water-surface elevation of about 1.5 ft at the flood peak.

The dredging alternatives for the bypass channel had little effect on calculated water-surface elevations at the peak of the Standard Project Flood. When sediment deposition was considered, there was a maximum difference of less than 0.5 ft in calculated water-surface elevations between the no-action case and either dredging alternative at the flood peak. This was attributed to the large quantity of material deposited during the rise of the flood.

Significant quantities of sediment will deposit in the first 0.5 mile of the proposed design channel downstream from the confluence with the bypass channel and in the existing pools at mile 12.47 and 13.35. About 32,000 cu yd can be expected on an annual basis if the channel is cleaned out every year. Sediment accumulation should be accounted for in design water-surface calculations, in the absence of an annual cleanout program.

Scour will occur in the bypass channel upstream from the confluence with Yellow Creek. This can be corrected with a head cut control structure. Otherwise, degradation may be accompanied by lateral migration of the stream, which could threaten existing levees and adjacent property.

Scour will also occur at the railroad bridge and at mile 13.02 where the design channel is constricted. Local scour protection should be provided for structures at these locations.

Recommendations

A 4-ft-deep low-flow channel with a base width of 25 ft upstream from Stony Fork and 40 ft downstream is recommended for the bypass channel. This was found to be the most sediment transport efficient cross section, based on the concept of channel-forming discharge and integration of the sediment transport and flow duration curves.

The nature of the channel bed in the vicinity of mile 11.85 should be thoroughly investigated. Evidence from this investigation indicates that the existing bed is composed of resistant material that controls stream response upstream. If the proposed cut exposes a less resistant layer of material, severe channel unraveling could occur. This can be corrected, if necessary, with a control structure.

For purposes of calculating design water-surface elevations, it is recommended that design invert elevations be increased to account for sediment deposition.

Based on decreases in composite roughness coefficients with channel widening, consideration should be given to lowering roughness coefficients in the design channel.

References

Ackers, P., and White, W. R. (1973). "Sediment transport: New approach and analysis," *Journal, Hydraulics Division, American Society of Civil Engineers* 99(HY11), 2041-2060.

Andrews, E. D. (1983). "Entrainment of gravel from naturally sorted riverbed material," *Geological Society of America Bulletin* 94, 1225-1231.

Burkham, Durl E., and Dawdy, David R. (1976). "Resistance equation for alluvial-channel flow," *Journal of the Hydraulics Division, ASCE* 102(HY10), 1479-1489.

Copeland, Ronald R. (1990). "Waimea river sedimentation study, Kauai, Hawaii; Numerical model investigation," Technical Report HL-90-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Copeland, Ronald R., and Thomas, William A. (1989). "Corte Madera Creek sedimentation study; Numerical model investigation," Technical Report HL-89-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Laursen, Emmett M. (1958). "The total sediment load of streams," *Journal of the Hydraulics Division, ASCE* 84(HY1), 1530-1 to 1530-36.

Limerinos, J. T. (1970). "Determination of the Manning coefficient from measured bed roughness in natural channels," Geological Survey Water-Supply Paper 1989-B, U.S. Geological Survey, Washington, DC.

Meyer-Peter, E., and Müller, R. (1949). "Formulas for bed load transport," *Report on Second Meeting of International Association for Hydraulic Research*, Stockholm, Sweden, 39-64.

Paintal, A. S. (1971). "Concept of critical shear stress in loose boundary open channels," *Journal of Hydraulic Research* 9(1), 91-113.

Thomas, William A., Copeland, Ronald R., Raphelt, Nolan K., and McComas, Dinah N. "Hydraulic Design Package for Channels, SAM" (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Toffaleti, F. B. (1968). "A procedure for computation of the total river sand discharge and detailed distribution, bed to surface," Report 5, Committee on Channel Stabilization, Corps of Engineers, U.S. Army, printed at U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

U.S. Army Engineer Hydrologic Engineer Center. (1991). "HEC-6: Scour and deposition in rivers and reservoirs, users manual," Davis, CA.

_____. 1990. "HEC-2 water surface profiles user's manual," Davis, CA.

Yang, C. T. (1973). "Incipient motion and sediment transport," *Journal, Hydraulics Division, American Society of Civil Engineers* 99(HY10) 1679-1704

_____. (1984). "Unit stream power equation for gravel," *Journal, Hydraulics Division, American Society of Civil Engineers* 110(HY12), 1783-1797.

Table 1
Calculated Accumulated Deposition Between Miles 11.53 and 14.96

End of Year	Existing Channel cu yd	Design Channel cu yd	Attributed to Design, cu yd
1	4,000	34,200	30,200
2	6,600	36,700	30,100
3	5,900	37,700	31,800
4	6,400	39,000	32,600
5	8,200	37,900	29,700
6	9,100	40,700	31,600
7	7,200	39,200	32,000
8	8,100	39,800	31,700
9	8,400	44,900	36,500
10	8,200	41,600	33,400

Table 2
Calculated Water-Surface Elevations for Standard Project
Flood Peak Discharge

Mile	Base Test		Alternative 1		Alternative 2	
	Without Sediment	With Sediment	Without Sediment	With Sediment	Without Sediment	With Sediment
3.89	1170.1	1170.0	1170.2	1170.2	1170.2	1170.2
3.80	1166.1	1166.3	1166.0	1166.0	1166.0	1166.1
3.63	1163.9	1165.4	1163.7	1164.9	1163.5	1164.7
3.48	1163.6	1164.5	1163.4	1164.0	1163.3	1163.9
3.32	1163.4	1164.2	1163.3	1163.9	1163.2	1163.7
3.19	1163.2	1164.0	1163.2	1163.8	1163.1	1163.7
2.90	1162.6	1162.6	1162.5	1162.5	1162.5	1162.4
2.75	1162.1	1162.1	1162.1	1162.0	1162.1	1162.0
2.50	1160.7	1160.6	1160.7	1160.6	1160.7	1160.6
2.26	1158.9	1158.9	1158.9	1158.8	1158.9	1158.8
2.10	1157.9	1157.9	1157.9	1157.8	1157.9	1157.9
1.70	1155.6	1155.6	1155.6	1155.6	1155.6	1155.6

Table 3
Adjustment to Design Invert Elevation to Account for Sediment Deposition

Cross Section	Original Design Invert Elevation	Calculated Bed Change ft	Recommended Invert Elevation
11.53	1101.3	-	1101.3
11.85	1101.7	-	1101.7
12.13	1102.1	0.7	1102.8
12.47	1100.9	2.3	1103.2
12.80	1103.1	0.2	1103.3
13.02	1103.7	-	1103.7
13.20	1103.9	-	1103.9
13.35	1101.8	2.4	1104.2
13.50	1103.3	1.2	1104.5
13.70	1105.5	0.8	1106.3
13.90	1106.1	-	1106.1
14.15	1105.9	-	1105.9
14.35	1107.2	-	1107.2
14.61	1107.7	0.8	1108.5
14.78	1108.2	0.9	1109.1
14.96	1108.6	2.9	1111.5

Note: Elevations are given in feet NGVD.

Plate 1

DISCHARGE, CFS

8000.00 7000.00 6000.00 5000.00 4000.00 3000.00 2000.00 1000.00 0.00

TIME, DAYS

0.0   100.0   200.0   300.0   400.0   500.0   600.0   700.0   800.0

26 JAN 78

4 DEC 78

20 JAN 79

21 MAR 80

3 JAN 82

7 MAY 84

18 FEB 86

9 NOV 86

20 JAN 88

12 JAN 89

17 MAR 90

23 DEC 90

23 MAR 91

78 79 80 81 82 83 84 85 86 87 88 89 90 91
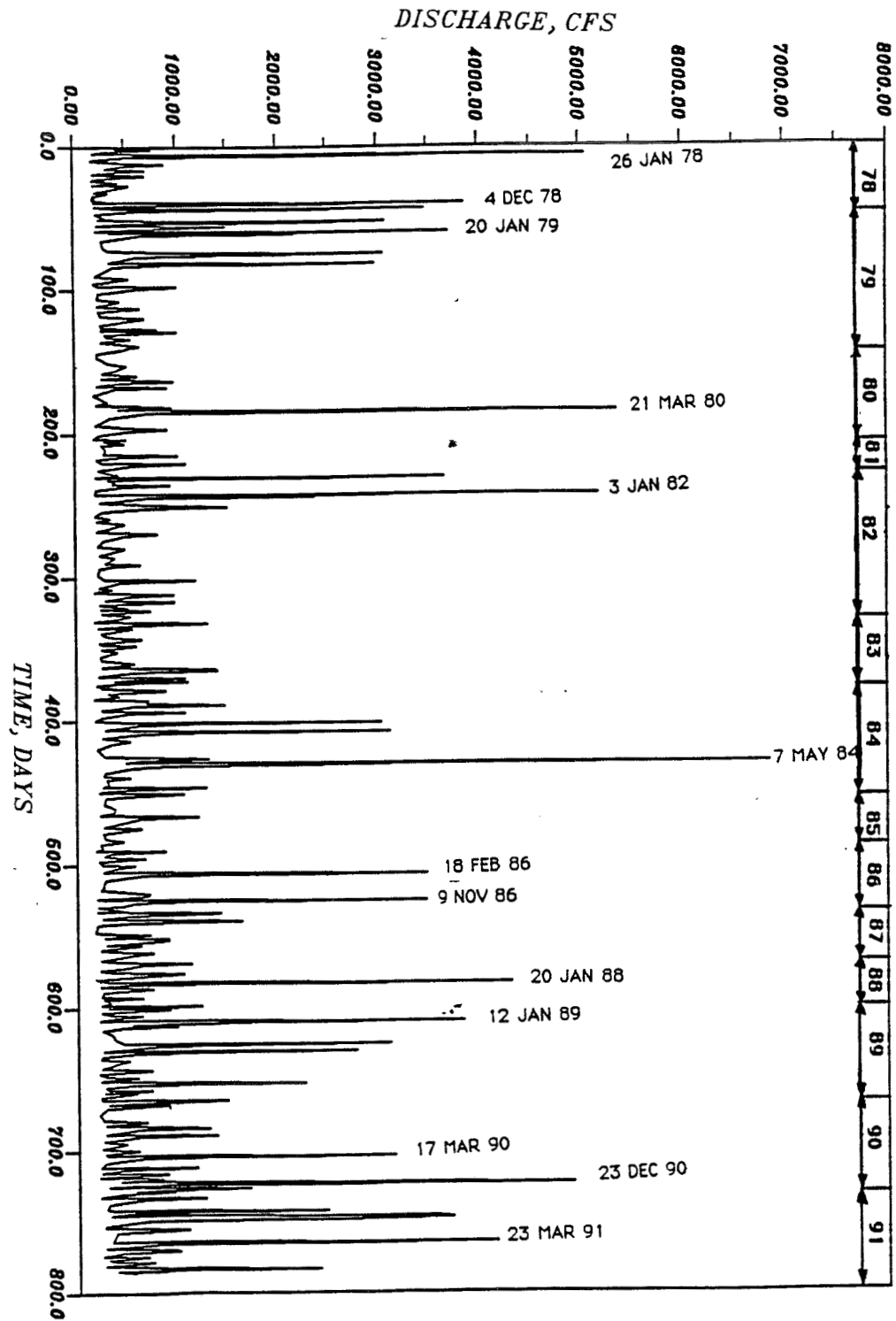
HISTORICAL HISTOGRAPH
YELLOW CREEK 1978-1991
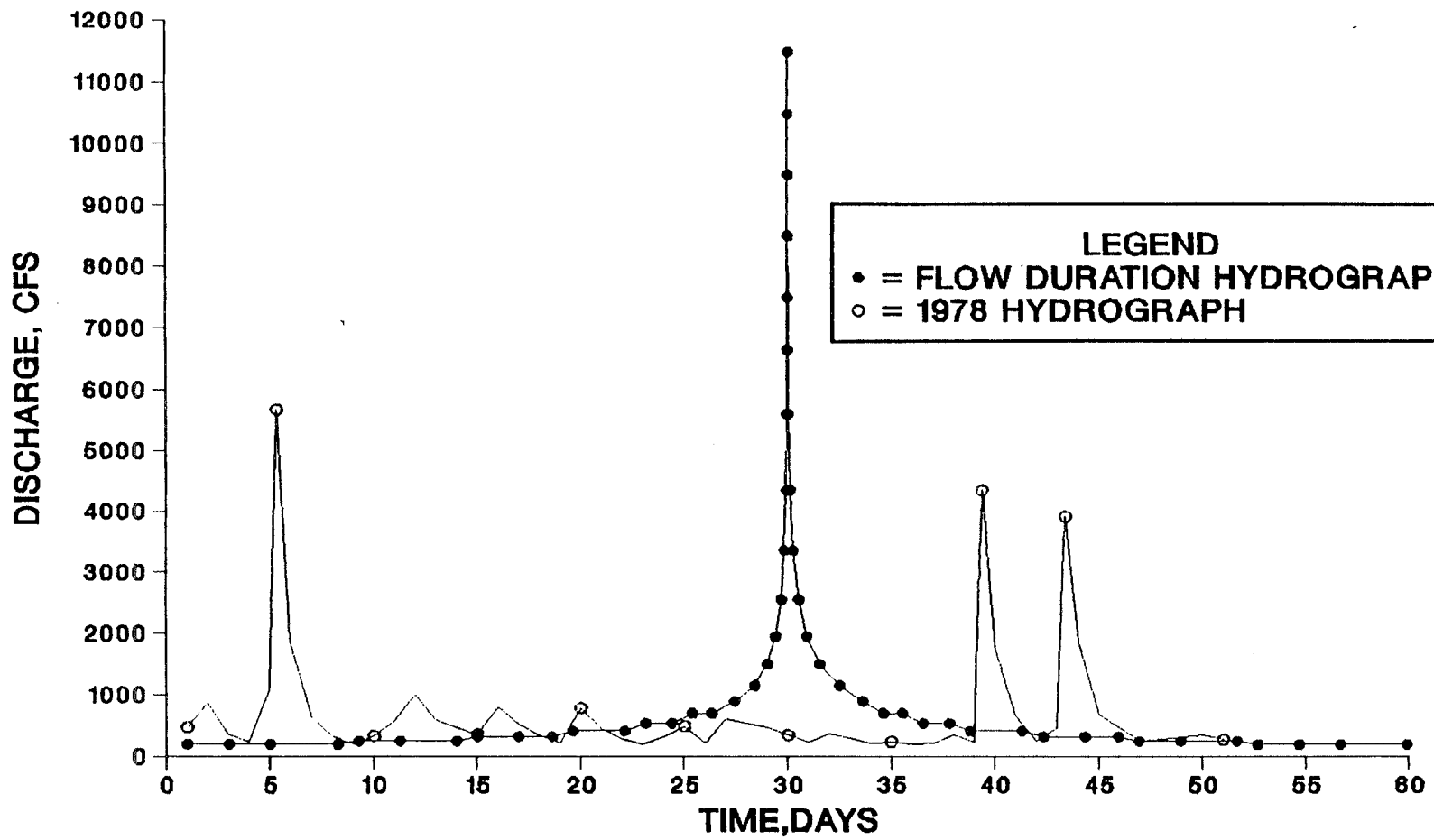
FLOW DURATION AND 1978 HYDROGRAPH

BED MATERIAL GRADATIONS
YELLOW CREEK CHANNEL AND BANKS

Plate 3

BED MATERIAL GRADATIONS
BYPASS CHANNEL DOWNSTREAM FROM CHUTE
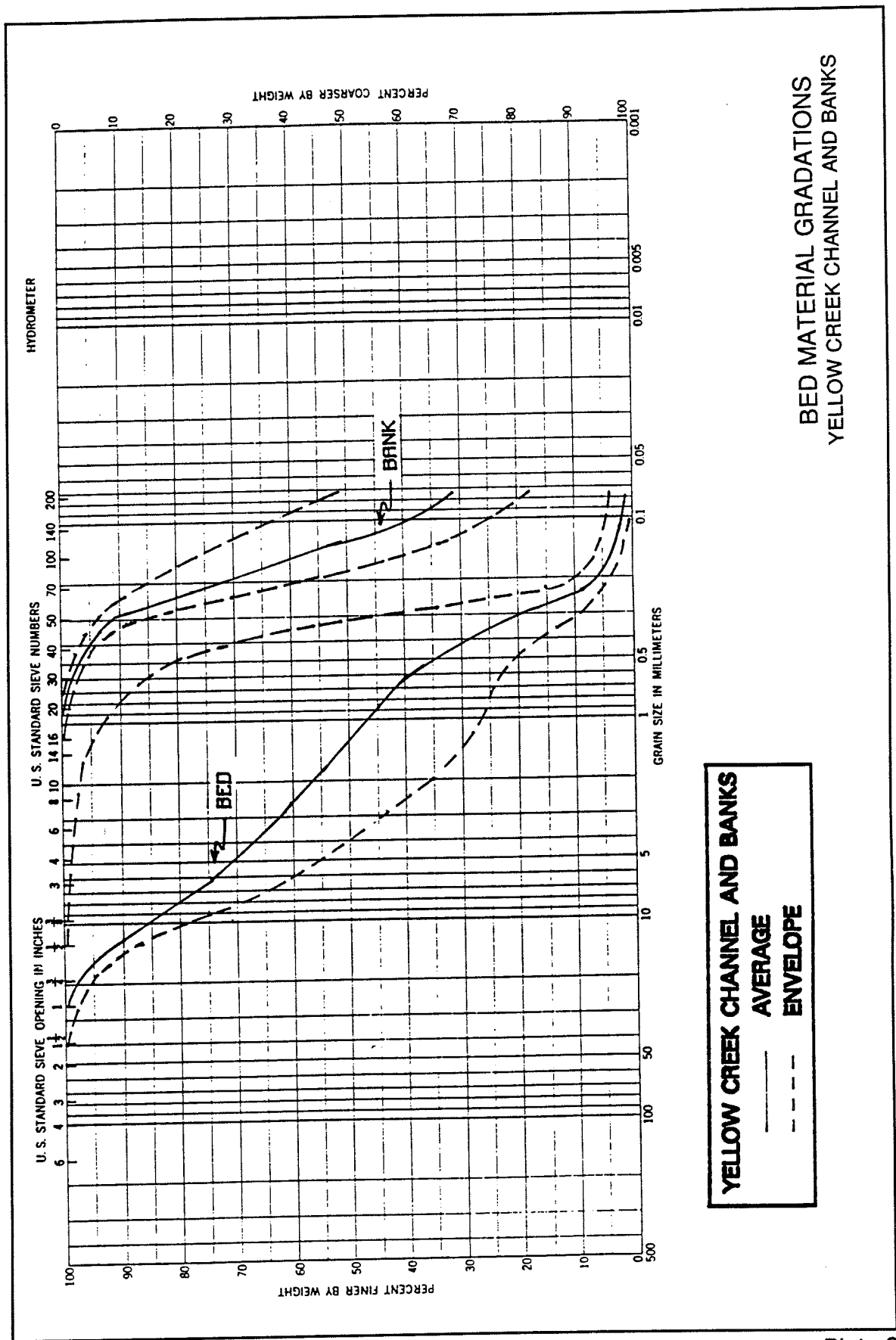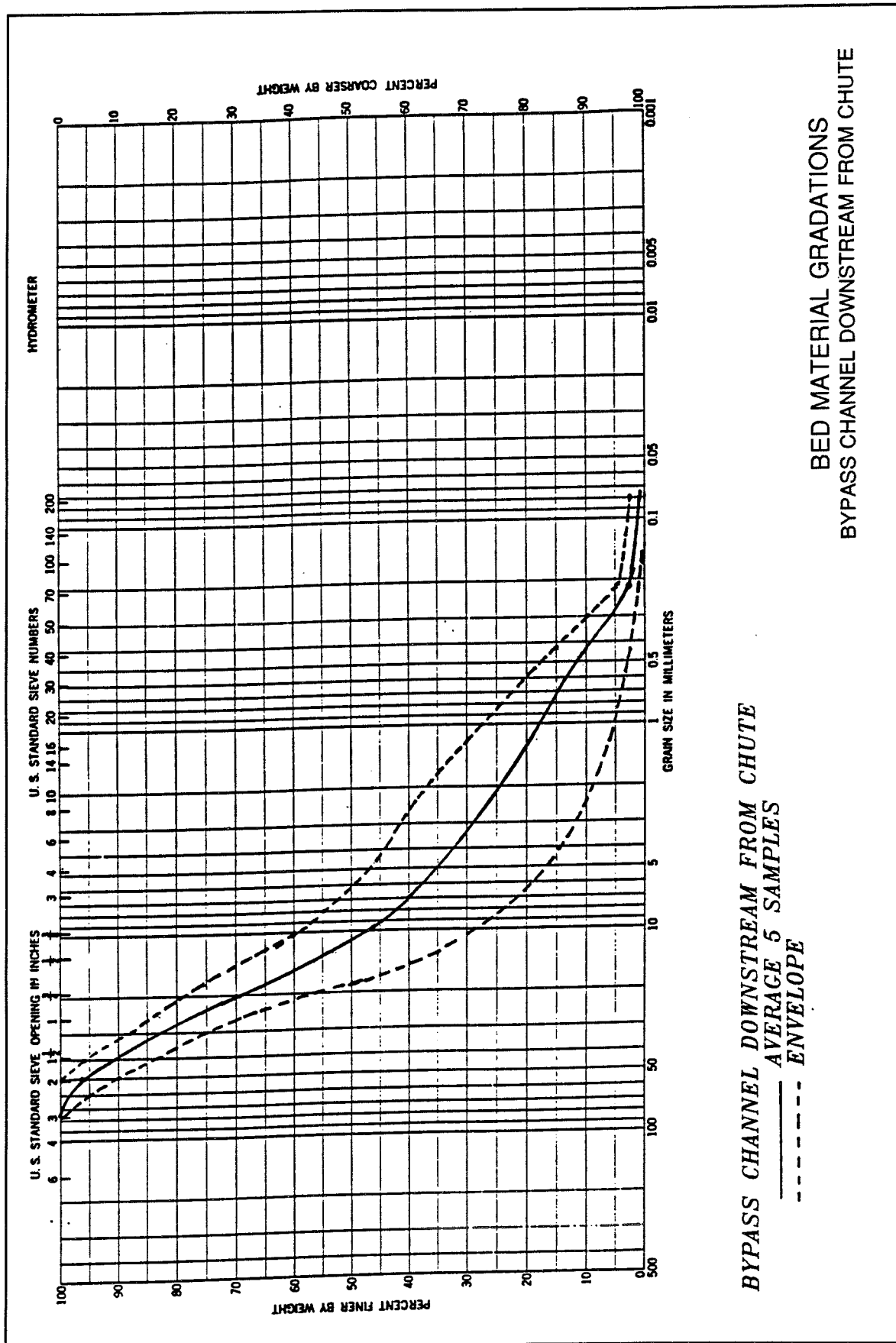
BYPASS CHANNEL DOWNSTREAM FROM CHUTE
AVERAGE 5 SAMPLES
———— ENVELOPE

Plate 4
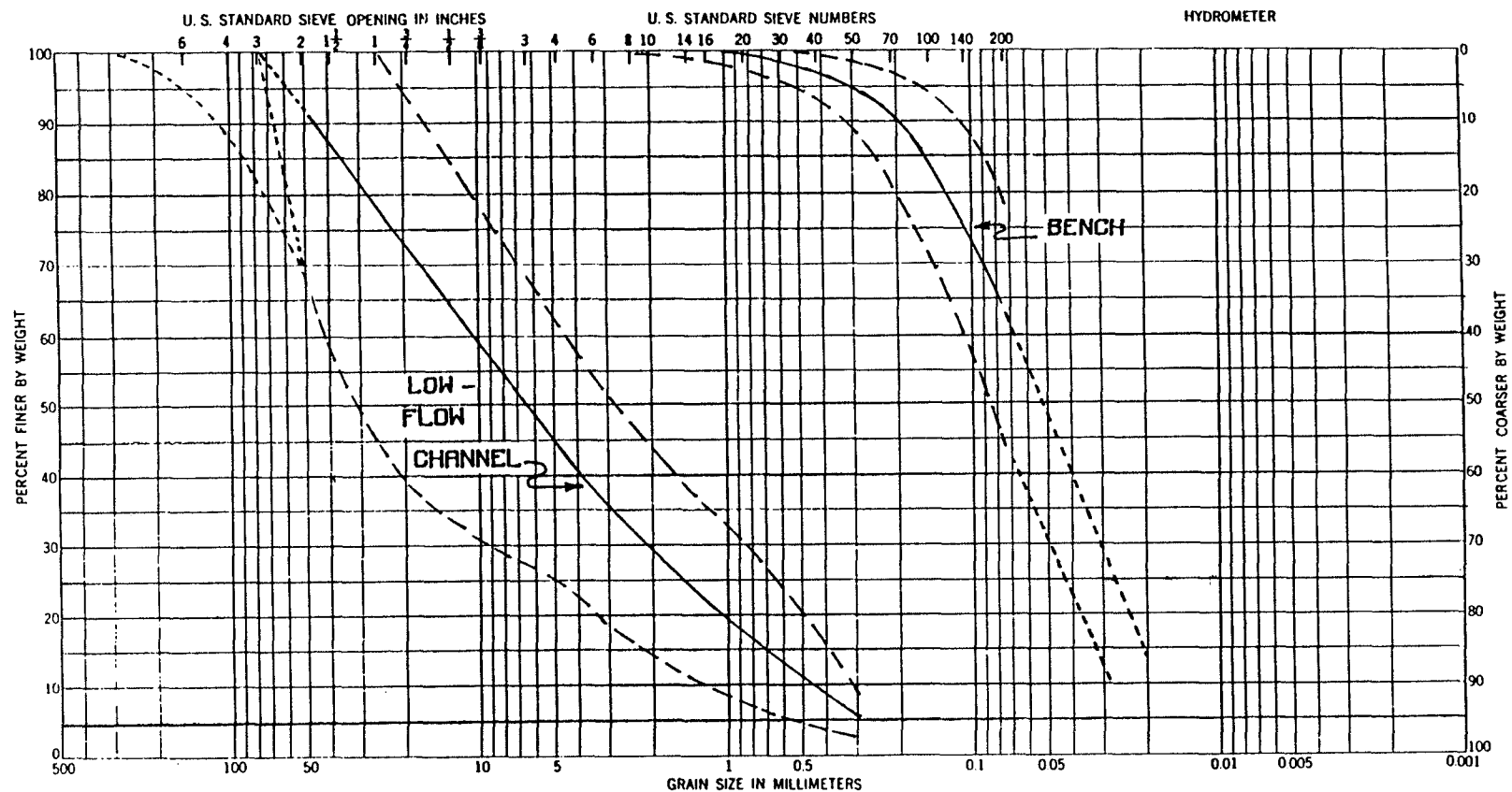
BED MATERIAL GRADATIONS
BYPASS CHANNEL UPSTREAM FROM CHUTE

Plate 5

LEGEND
□ = *TOFFALETI*
○ = *YANG*
△ = *ACKERS–WHITE*
+ = *MEYER–PETER/MÜLLER*
× = *LAURSEN–MADDEN*
◇ = *LAURSEN–COPELAND*
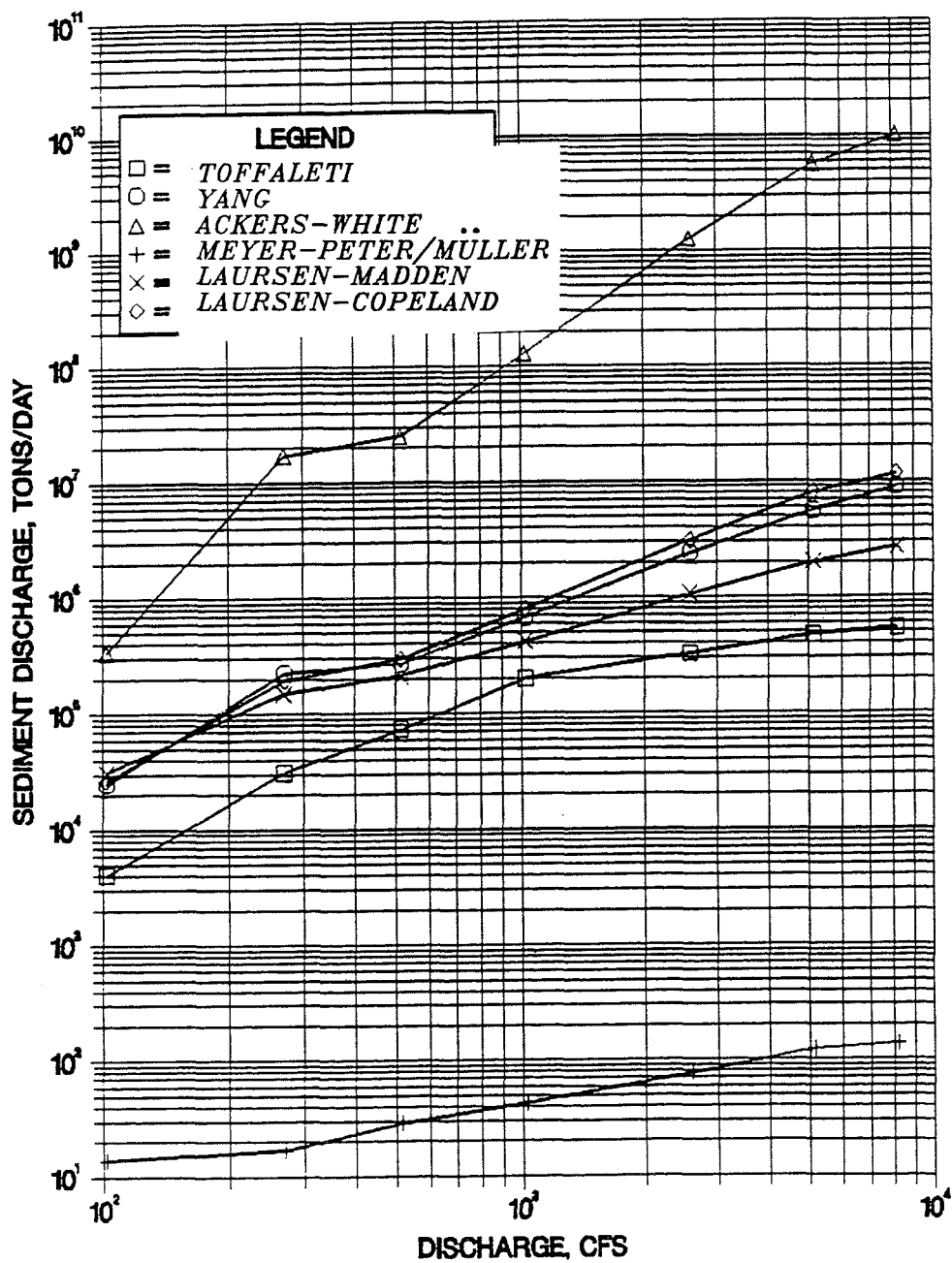
CALCULATED SEDIMENT TRANSPORT POTENTIAL FOR VERY
FINE SAND AT UPSTREAM MODEL BOUNDARY

Plate 6

FUNCTION (U* PRIME/ω) FOR SEDIMENT
TRANSPORT FUNCTION

LEGEND

□ FLUME DATA

○ RIVER DATA

Plate 7

ADJUSTED TOTAL SAND
INFLOW TO MODEL

TOTAL SEDIMENT DISCHARGE, TONS/DAY

DISCHARGE, CFS

ADJUSTED TOTAL SEDIMENT
INFLOW TO MODEL
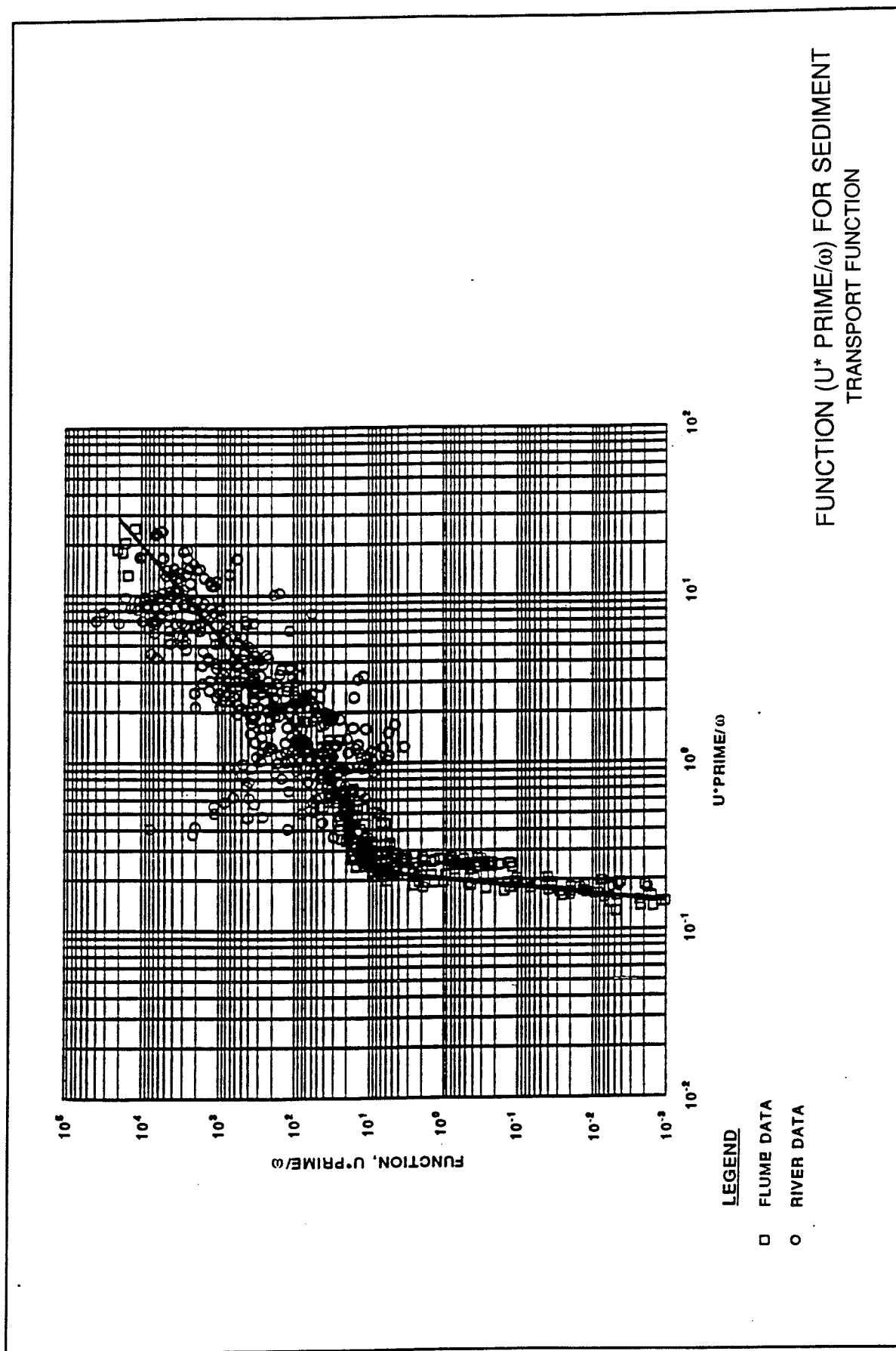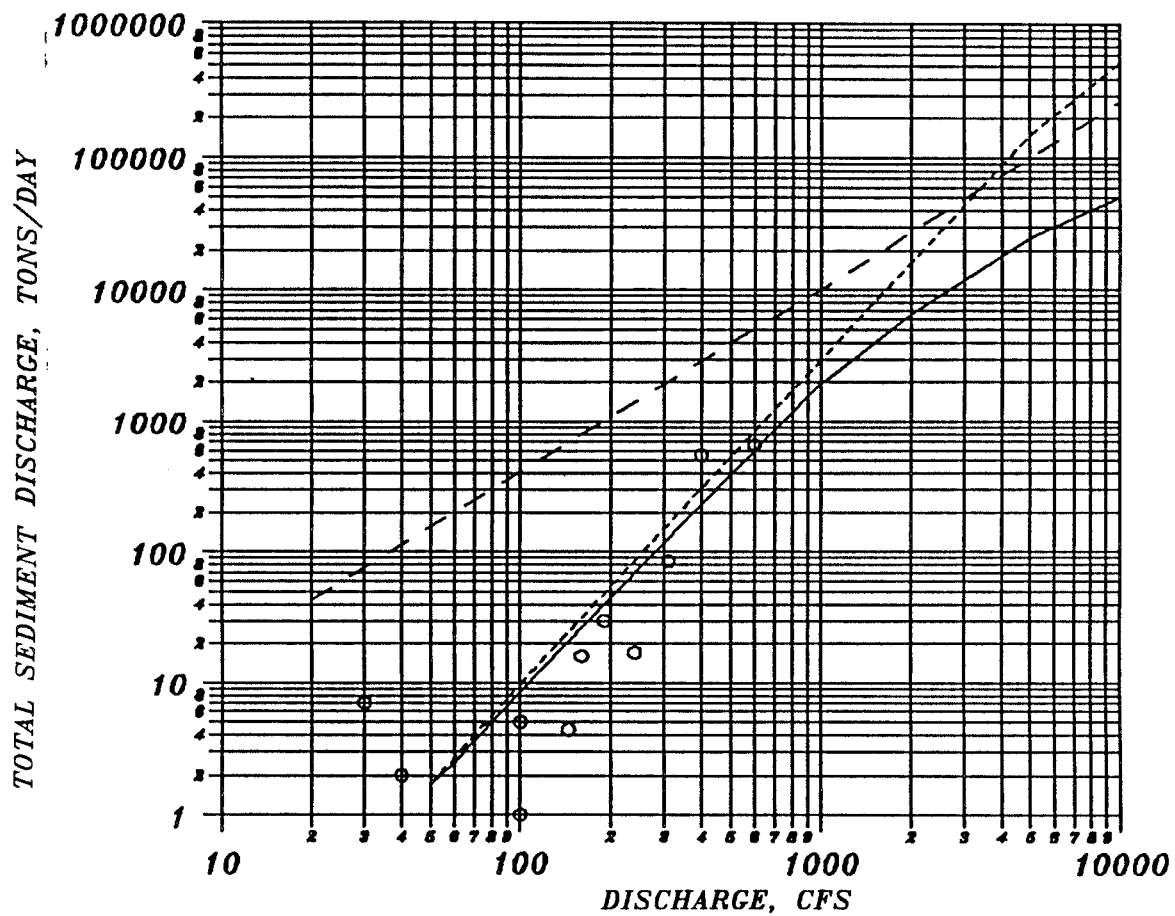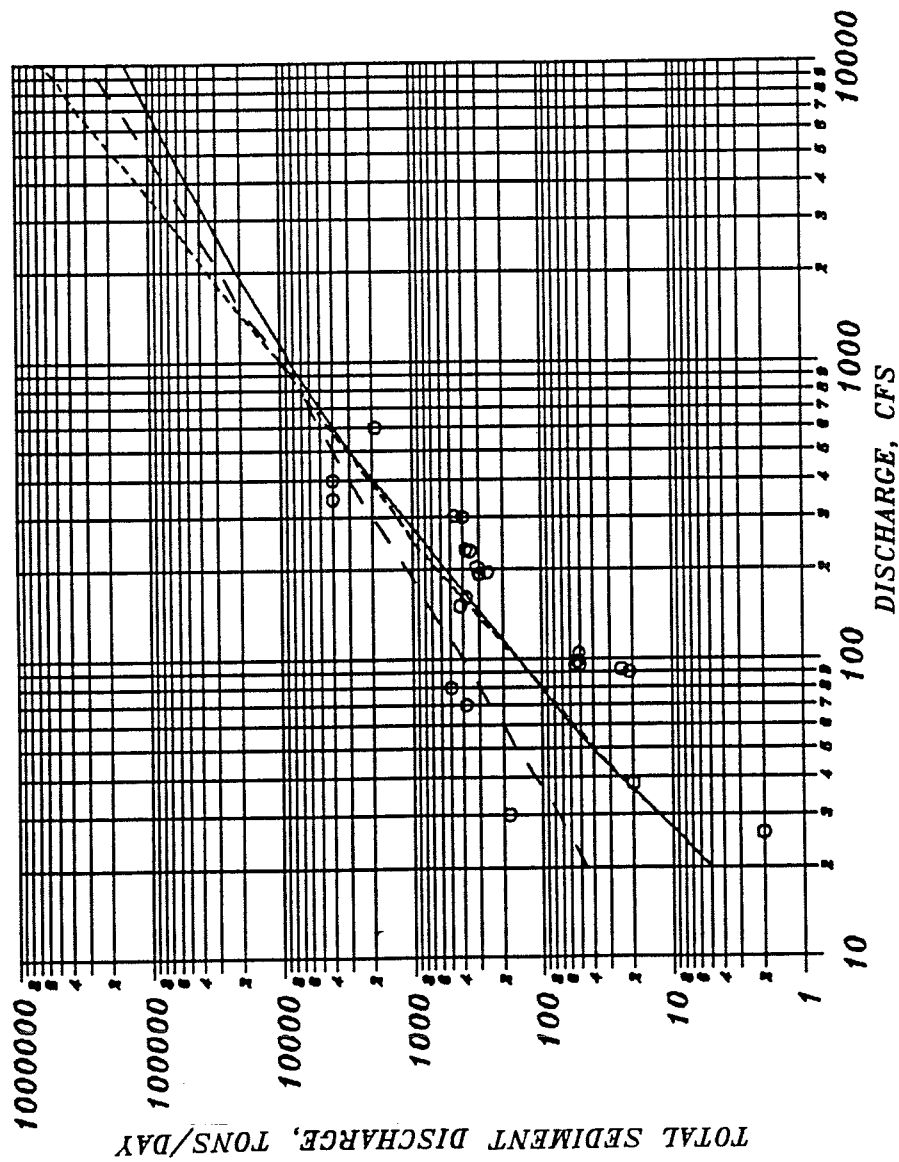
LEGEND

— TOFFALETI/MEYER–PETER AND MÜLLER
---- LAURSEN–COPELAND
– – SINGLE GRAIN SIZE (VFS)
ooooo SAMPLED DATA

Plate 9

DISTANCE FROM CHANNEL CENTER LINE, FT

SOIL STRATUM, BYPASS CHANNEL
MILE 0.5

NOTE: BASED ON 1937 TEST BORINGS

Plate 10

FLOW DURATION
AVERAGE DAILY FLOWS
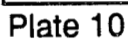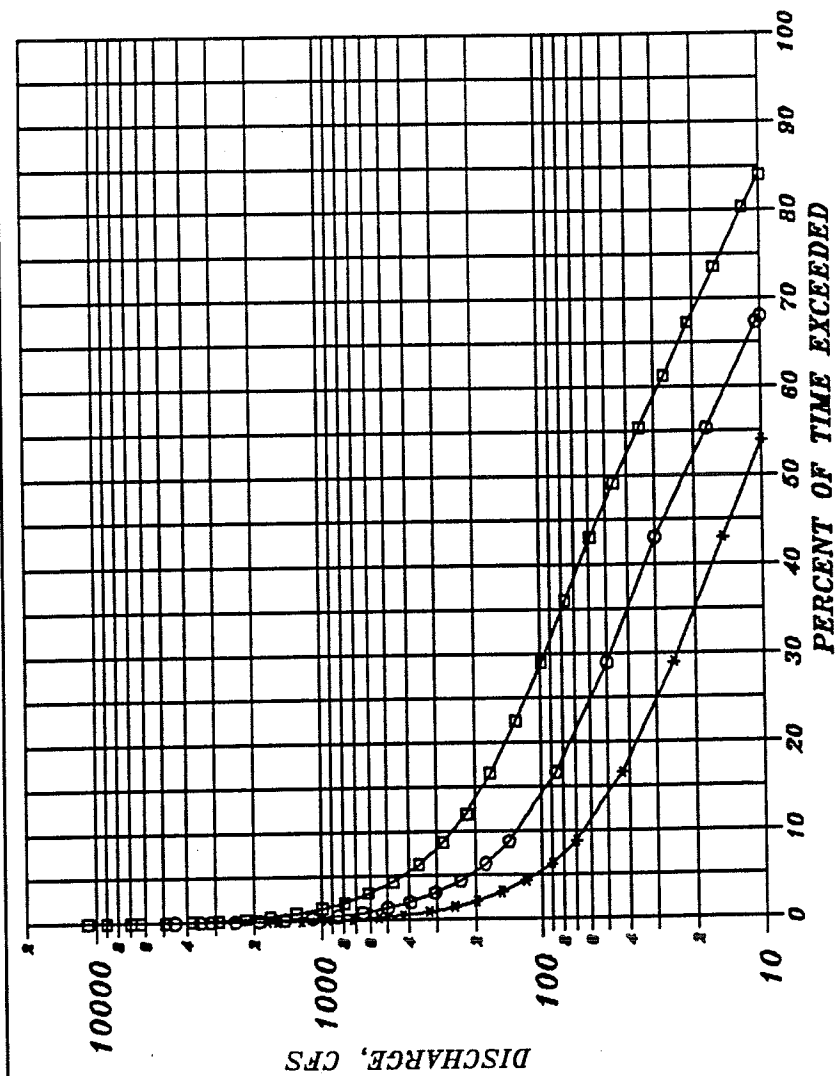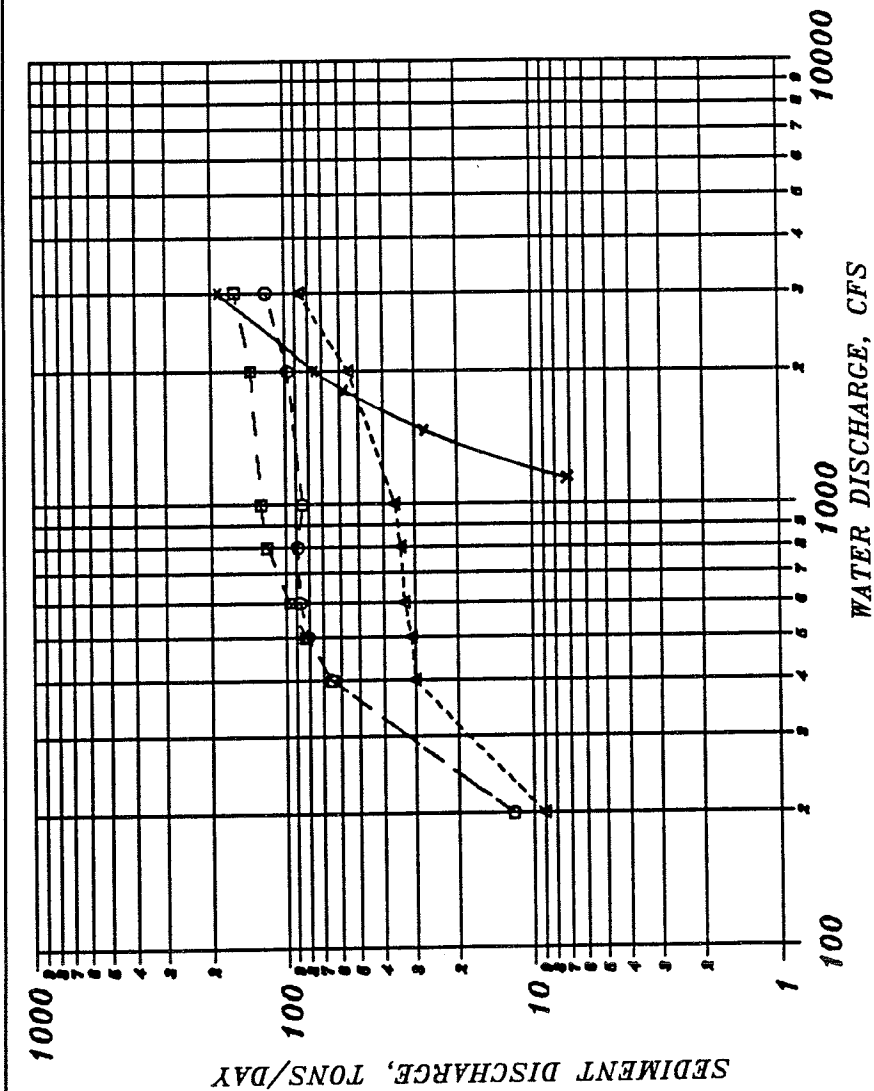
LEGEND

BENNETTS FORK
BYPASS CHANNEL AT CHUTE
YELLOW CREEK AT GAGE

Plate 11

SEDIMENT TRANSPORT CAPACITY
MEYER-PETER/MÜLLER, MILE 3.39-3.13

LEGEND
TRAPEZOIDAL CHANNEL
2-FT LOW–FLOW CHANNEL
4-FT LOW–FLOW CHANNEL
6-FT LOW–FLOW CHANNEL

Plate 12

SEDIMENT TRANSPORT CAPACITY
LAURSEN-COPELAND, MILE 3.39 - 3.13
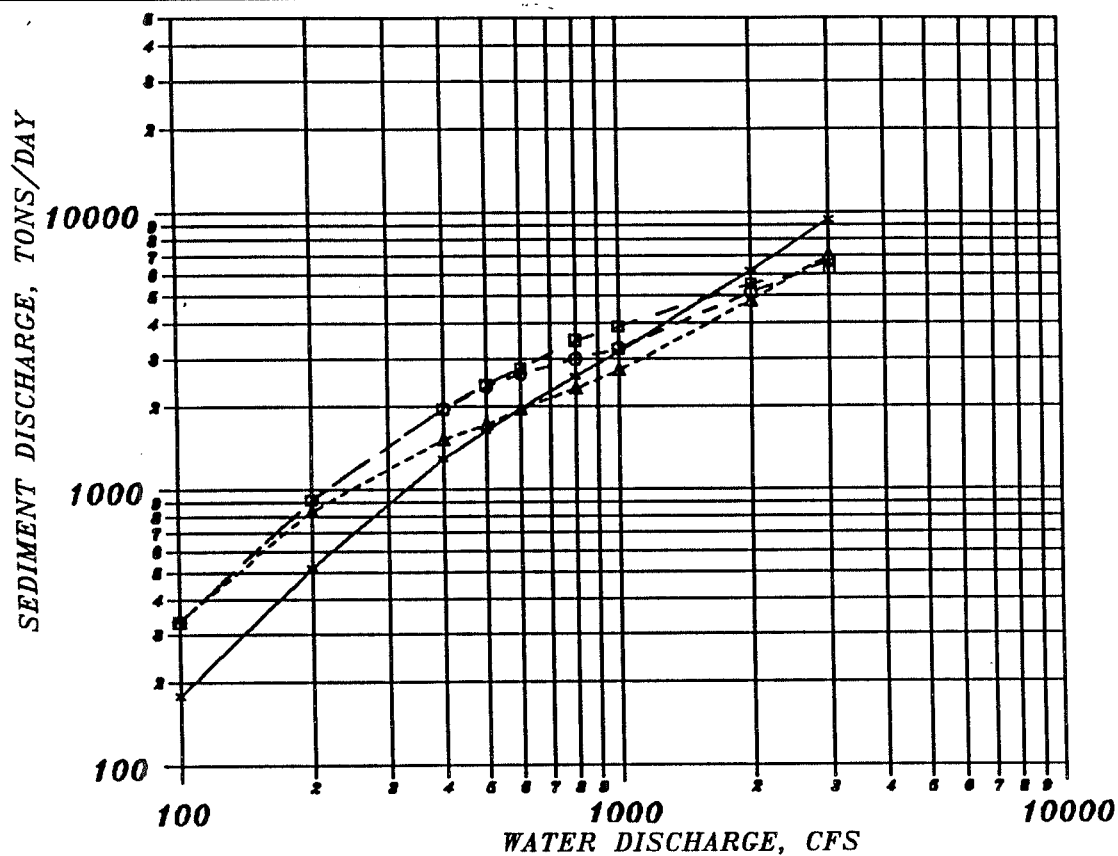
Plate 13

Plate 14

SEDIMENT DISCHARGE, TONS/DAY

WATER DISCHARGE, CFS

LEGEND
+++++ TRAPEZOIDAL CHANNEL
▲▲▲▲▲ 2-FT LOW-FLOW CHANNEL
ⓞⓞⓞⓞⓞ 4-FT LOW-FLOW CHANNEL
▣▣▣▣▣ 6-FT LOW-FLOW CHANNEL

SEDIMENT TRANSPORT CAPACITY
TOFFALETI/MEYER-PETER/MÜLLER, MILE 3.39-3.13
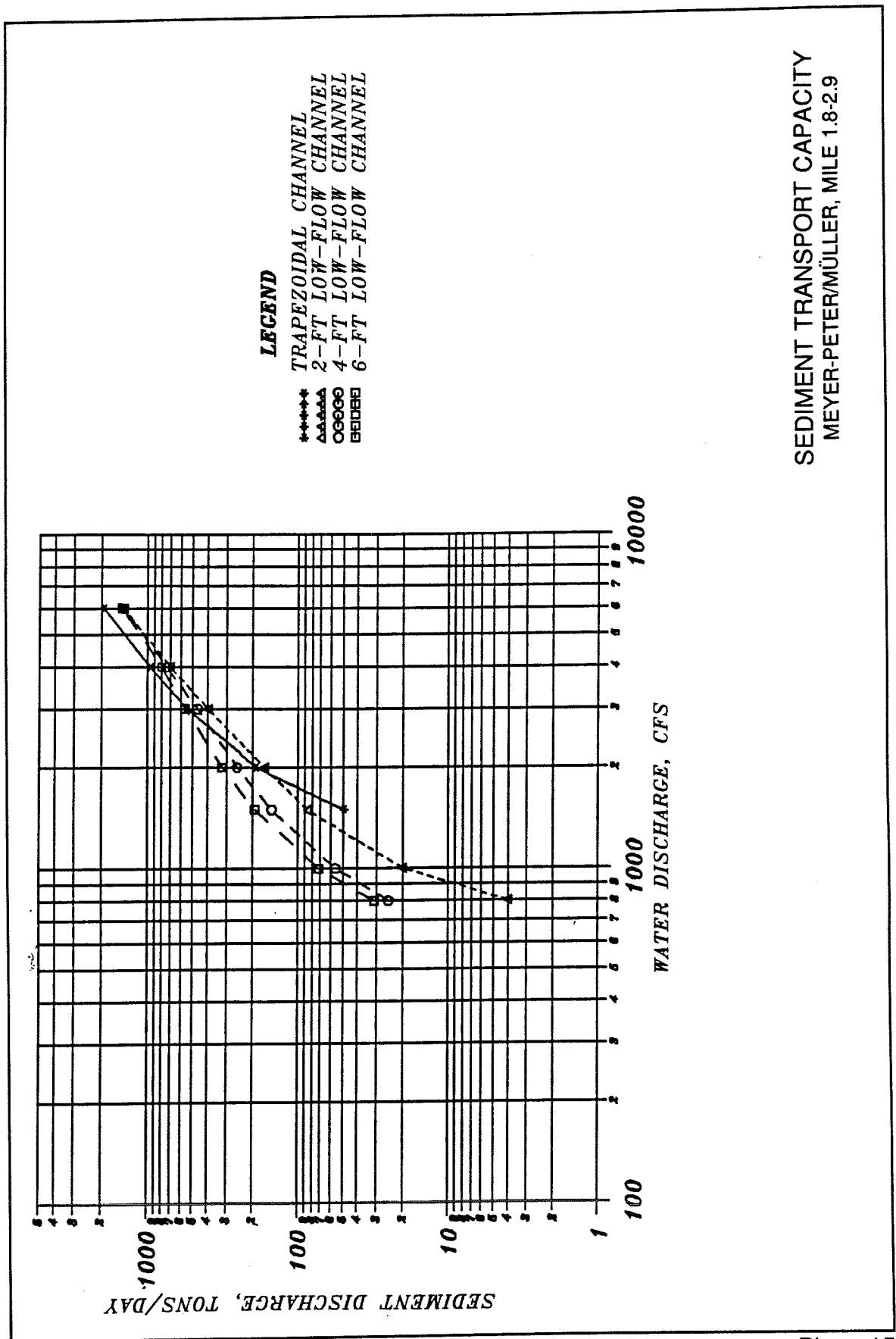
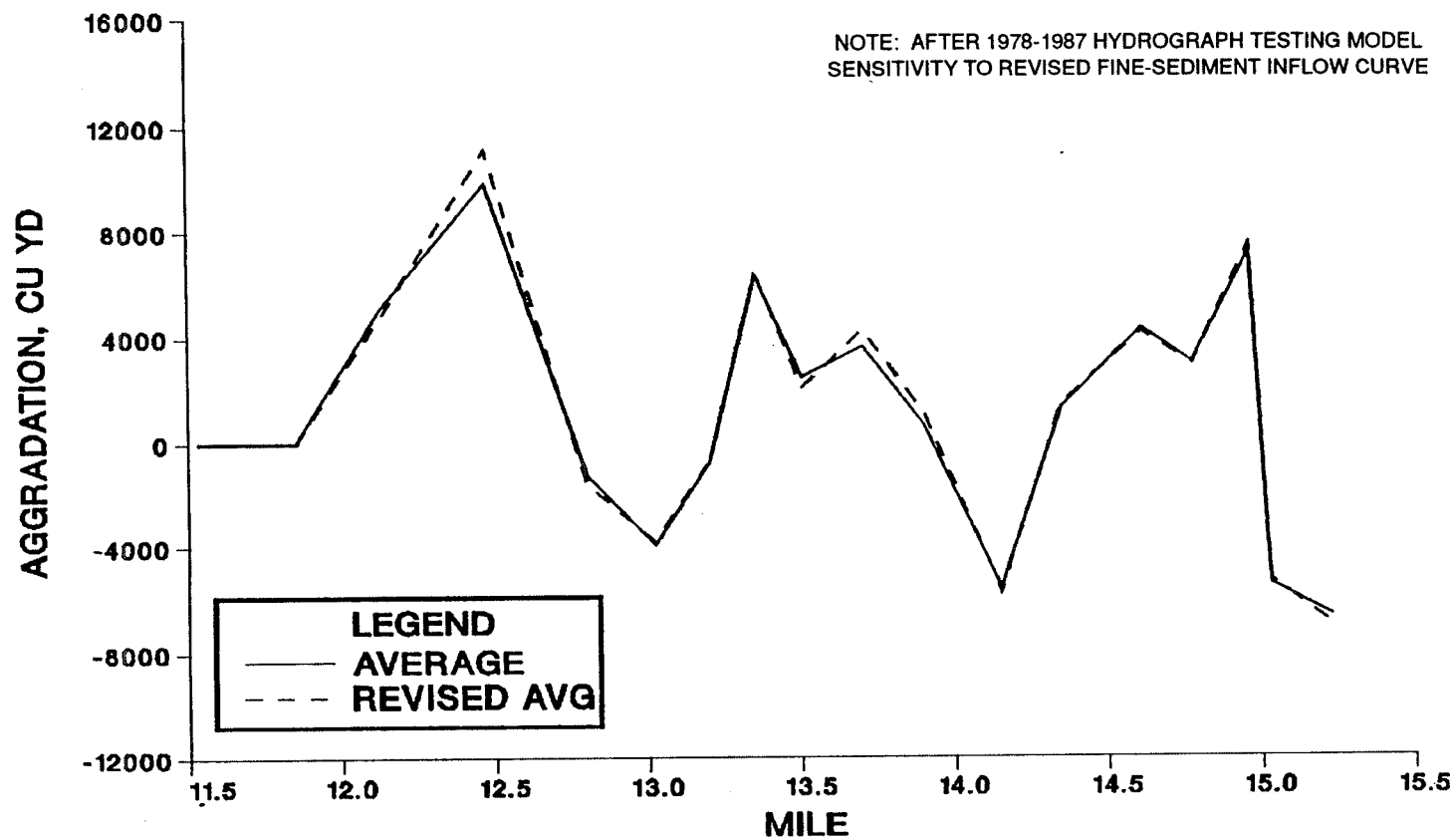SEDIMENT TRANSPORT CAPACITY
MEYER-PETER/MÜLLER, MILE 1.8-2.9
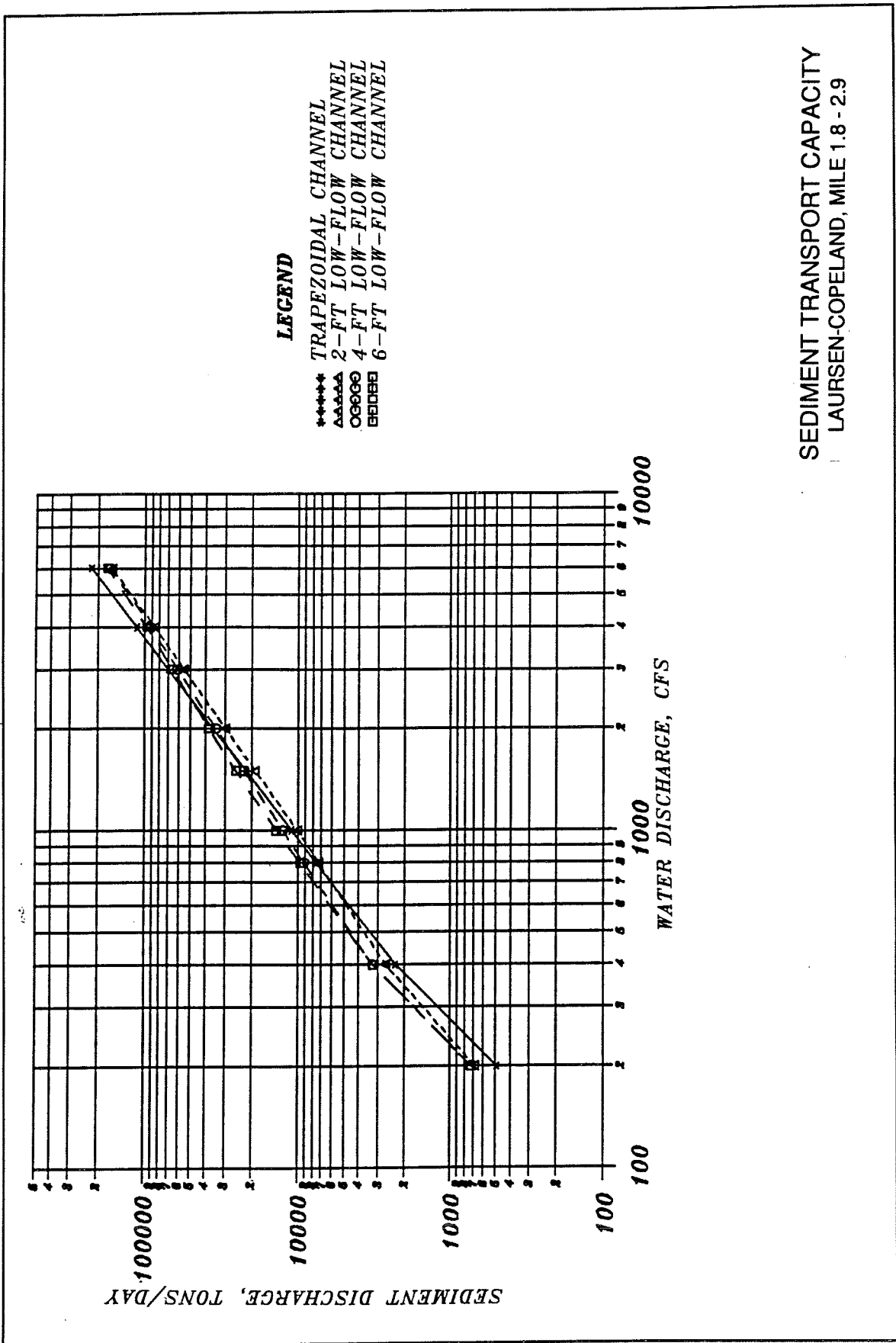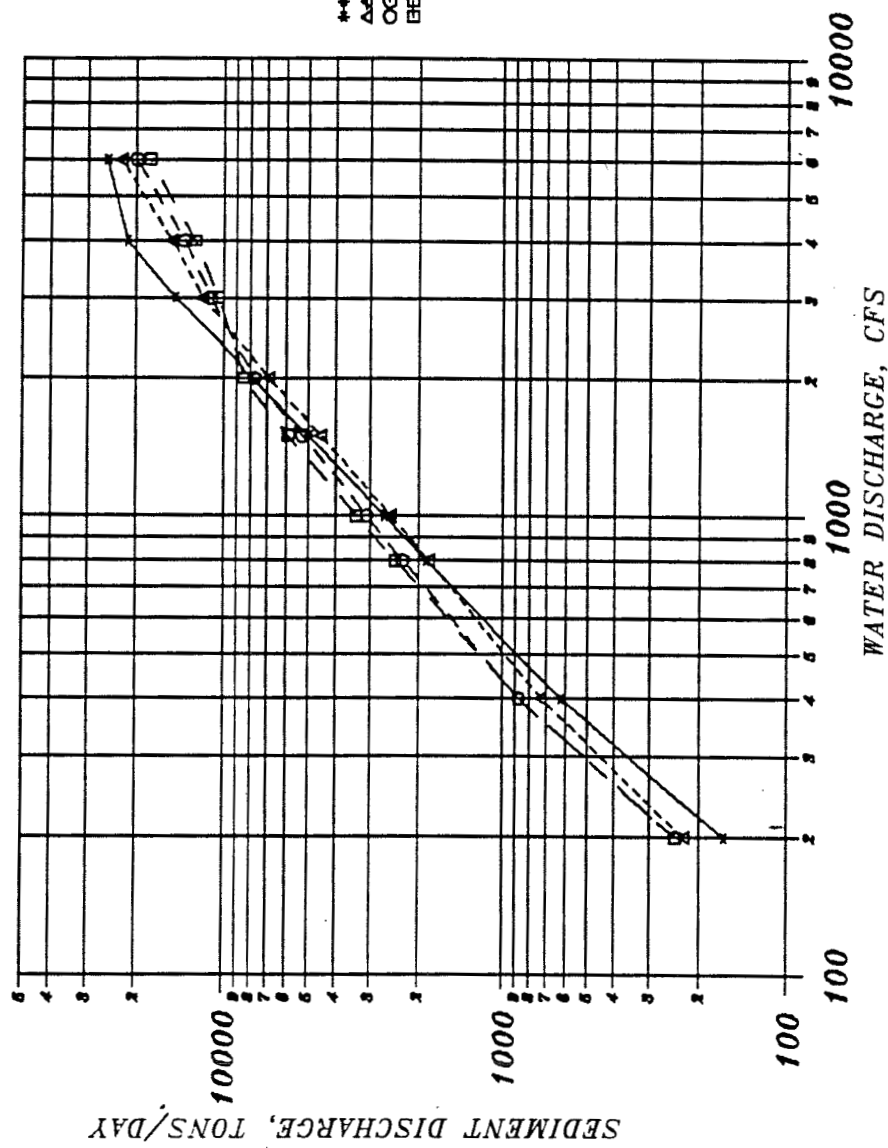
Plate 15

SEDIMENT TRANSPORT CAPACITY
LAURSEN-COPELAND, MILE 1.8 - 2.9

LEGEND
+++++ TRAPEZOIDAL CHANNEL
▲▲▲▲▲ 2-FT LOW-FLOW CHANNEL
ⓞⓞⓞⓞⓞ 4-FT LOW-FLOW CHANNEL
⊟⊟⊟⊟⊟ 6-FT LOW-FLOW CHANNEL

WATER DISCHARGE, CFS

SEDIMENT DISCHARGE, TONS/DAY

Plate 16

SEDIMENT TRANSPORT CAPACITY
TOFFALETI/MEYER-PETER/MÜLLER, MILE 1.8-2.9

Plate 17

SEDIMENT INFLOW CURVES

Plate 18

NOTE: AFTER 1978-1987 HYDROGRAPH TESTING MODEL
SENSITIVITY TO REVISED FINE-SEDIMENT INFLOW CURVE

DIFFERENCE IN CALCULATED SEDIMENT ACCUMULATION
BY REACH, BETWEEN EXISTING AND DESIGN CHANNELS

Plate 19
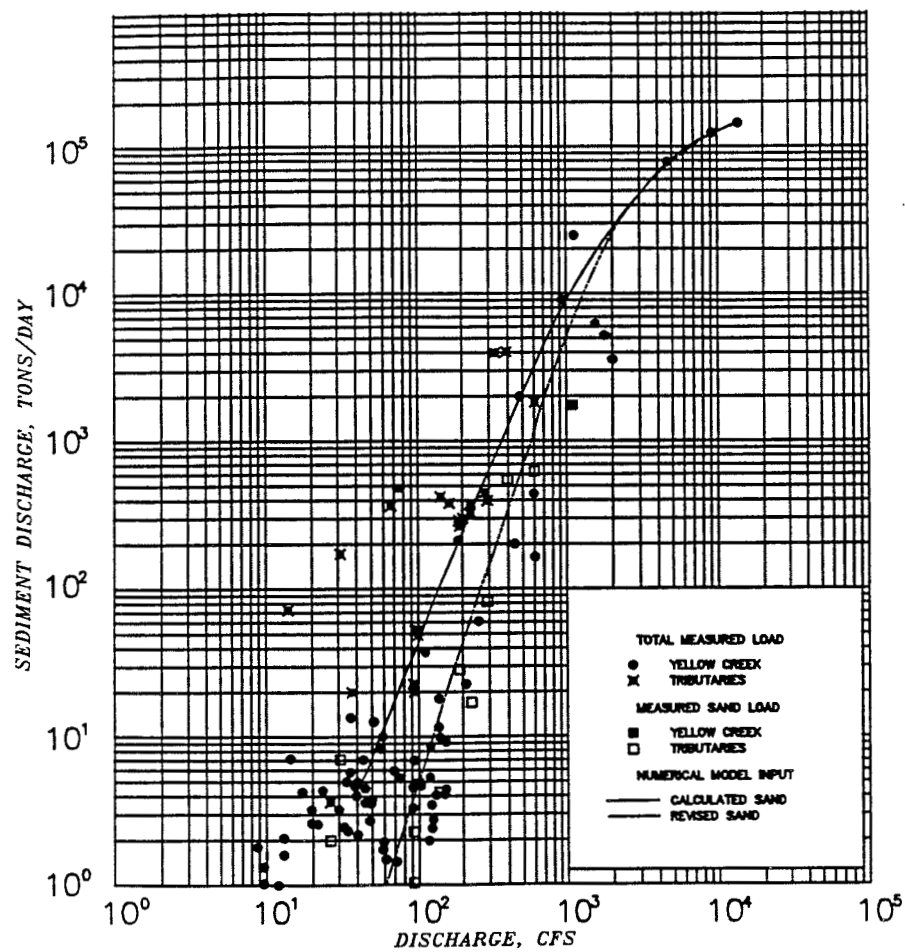
Appendix A
Description of TABS-1
Computer Program

The computer program TABS-1 calculates water-surface profiles and changes in the streambed profile. Water velocity, water depth, energy slope, sediment load, gradation of the sediment load, and gradation of the bed surface are also computed. Water-surface profile and sediment movement calculations are fully coupled using an explicit computation scheme. First, the conservation of energy equation is solved to determine the water-surface profile and pertinent hydraulic parameters (velocity, depth, width, and slope) at each cross section along the study reach:

$$\frac{\partial H}{\partial X} + \frac{\partial \left(\alpha \, \frac{V^2}{2g} \right)}{\partial X} = S \tag{A1}$$

where

H = water-surface elevation
X = direction of flow
α = coefficient for the horizontal distribution of velocity
V = average flow velocity
g = acceleration due to gravity
S = slope of energy line

In addition, the continuity of sediment material is expressed by

$$\frac{\partial G}{\partial X} + B \cdot \frac{\partial y_s}{\partial t} = q_s \tag{A2}$$

where

G = rate of sediment movement, cu ft/day
X = distance in direction of flow, ft
B = width of movable bed, ft
y_s = change in bed surface elevation, ft
t = time, days
q_s = lateral inflow of sediment, cu ft/ft/day

The third equation relates the rate of sediment movement to hydraulic parameters as follows:

$$G = f(V,y,B,S,T,d_{eff},d_{si},P_i) \qquad (A3)$$

where

y = effective depth of flow
T = water temperature
d_{eff} = effective grain size of sediment mixture
d_{si} = geometric mean of class interval
P_i = percentage of ith size class in the bed

The numerical technique used to solve Equation A1 is commonly called the Standard Step Method. Equation A2 has both time and space domains. An explicit form of a six-point finite difference scheme is utilized. Several equations of the form of Equation A3 are available. These transport capacity equations are empirical and G is determined analytically.

Equation A2 is the only explicit equation, but it controls the entire analysis by imposing stability constraints. Several different computation schemes were tested, and the six-point scheme proved the most stable. No stability criteria have been developed for this scheme. The rule of thumb is to observe the amount of bed change during a single computation interval and reduce the computation time until that bed change is tolerable.

Oscillation in the bed elevation is a key factor in selecting a suitable computation interval. The computation time interval must be made short enough to eliminate oscillation. On the other hand, computer time increases as the computation interval decreases. The proper value to use is determined by successive approximations, running test cases, and observing the amount of bed change.

Several supporting equations are required in transforming the field data for the computer analysis. The Manning equation is used to evaluate friction loss. Average geometric properties are combined, using an average end area approach, into an average conveyance for the reach. Manning's roughness coefficients are entered for the channel and both overbanks and may be

changed with distance along the channel, discharge, or stage. Construction and expansion losses are calculated as "other" losses by multiplying a coefficient times the change in velocity head. All geometric properties are calculated from cross-section coordinates.

Only subcritical flow may be analyzed in the computer program; however, zones of critical or supercritical flow may occur within the study reach. The program treats supercritical zones as "critical" for determination of water-surface elevation, but calculates hydraulic parameters for sediment transport based on normal depth. Critical depth in a section with both channel and overbank is defined as the minimum specific energy for that section assuming a level water surface. Starting water-surface elevations can be input as a rating curve with stage and discharge, or stage can be set for each specific time interval. Steady-state conditions are assumed for each time interval, although the discharge may be changed to account for tributary inflow. A hydrograph is simulated by creating a histograph of steady-state discharges, using small time intervals when discharge variations are great and longer time intervals when changes in water and sediment discharges are small.

In some cases the temperature of water can be an important parameter in sediment transport and, consequently, may be prescribed with each water discharge in the hydrograph. Flexibility of input permits a value to be entered as needed to change from a previous entry.

Geometry is input into the numerical model as a series of cross sections similar to the widely used HEC-2 backwater program (US Army Engineer Hydrologic Engineering Center 1990[1]). A portion of the cross section is designated as movable and a dredging template may also be specified. Spacing of cross sections is somewhat more critical for TABS-1 than it is for HEC-2 because of numerical stability problems. Long reach lengths are desirable because reach length and computation interval are related. Very short time intervals may be required if excessive bed changes occur within a specific reach. No special provisions are available to calculate head losses at bridges. The contracted opening may be modeled such that scour and deposition are simulated during the passing of a flood event, but calculated results must be interpreted with the aid of a great deal of engineering judgment and sensitivity analysis.

Four different sediment properties are required: (a) the total concentration of suspended and bed loads, (b) grain-size distribution for the total concentration, (c) grain-size distribution for sediment in the streambed, and (d) unit weight of deposits. A wide range of sediment material may be accommodated in the transport calculations (0.004 mm to 64 mm).

The usefulness of a calculation technique depends a great deal upon the coefficients which must be supplied. As in HEC-2, Manning's n values, contraction coefficients, and expansion coefficients must be provided to

---

[1] References cited in this appendix are included in the References at the end of the main text.

accomplish the water-surface profile calculations. Several other coefficients are required for sediment calculations as follows:

a. The specific gravity and shape of sediment particles must be specified.

b. The bed shear stress at which silt or clay particles begin to move and deposit are required coefficients.

c. The unit weight of silt, clay, and sand deposits is somewhat like a coefficient because of the difficulty in measuring. Also, the density changes with time.

All of the sediment-related coefficients have default values because sediment data seem to be much more scarce than hydraulic data. There are fewer sources for generalized coefficients. All of the default values should be replaced by field data where possible, and the input data are structured for such a process.

13. ABSTRACT (Maximum 200 words)

Yellow Creek is located in the southeastern Kentucky coal mining region. Its major tributaries, Stony Fork and Bennetts Fork, are steep mountain streams that rise in the Cumberland Mountains near the Kentucky-Tennessee-Virginia tri-state corner and converge in a bowl-shaped valley that contains the city of Middlesboro, KY. In 1939, the U.S. Army Corps of Engineers completed construction of a 3.9-mile bypass channel around the town. Coal strip mines and spoil areas in the rugged hills have been the source of heavy sediment loads that have historically deposited in the flat-bottomed, trapezoidal bypass channel. Sediment aggradation has been so great that maintenance dredging has been conducted at least four times in the past, the most recent being in 1978-79.

Flood damage reduction alternatives for Middlesboro are currently being evaluated. One structural alternative is an enlarged channel immediately downstream of the city. A one-dimensional numerical model (TABS-1) was used to evaluate (a) dredging options in the existing bypass channel upstream from Middlesboro; and (b) potential aggradation and/or degradation in the proposed improved channel downstream from Middlesboro. The numerical model was adjusted to simulate measured aggradation in the upstream reaches of

(Continued)

14. SUBJECT TERMS Channel maintenance dredging Sedimentation Flood control Stable channel design Numerical model Yellow Creek	15. NUMBER OF PAGES 76
	16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT UNCLASSIFIED	18. SECURITY CLASSIFICATION OF THIS PAGE UNCLASSIFIED	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT

13. (Concluded).

the project and to simulate conditions in the existing downstream reaches of the project where there is no apparent aggradation or degradation trend. Alternative dredging cross-section options were determined using a new numerical model for hydraulic design called SAM. The design channel was then incorporated into the TABS-1 numerical model for evaluation. Maintenance requirements for the bypass channel were determined, and zones of aggradation and degradation in the improved channel reaches were identified. Proposed modifications to the improved channel design were tested in the numerical model. A more efficient design in terms of sediment transport was recommended. The original project design flood profile was also checked using the stable channel template. Appendix A describes the TABS-1 computer program.